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Waterways Experiment Station

Demonstration Erosion Control Project Monitoring Program

Fiscal Year 1994 Report

by Thomas J. Pokrefke, Nolan K. Raphelt, David L. Derrick, Billy E. Johnson, Michael J. Trawle, WES

Chester C. Watson, Colorado State University

Prepared for U.S. Army Engineer District, Vicksburg

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U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Chester C. Watson

Civil Engineering Department
Engineering Research Center
Colorado State University
Fort Collins, CO 80523

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Prepared for U.S. Army Engineer District, Vicksburg
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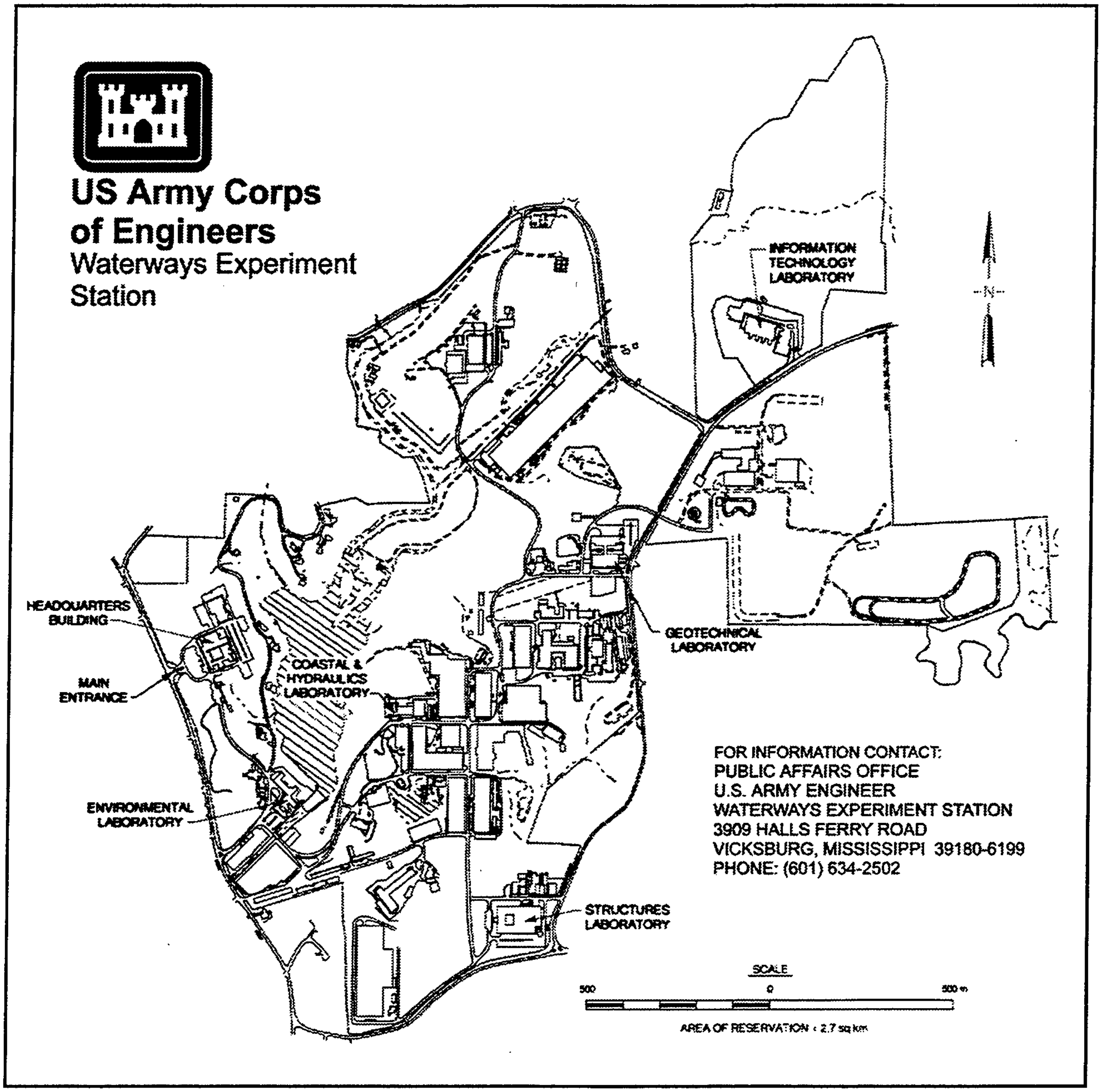

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Contents

Preface

This report discusses work performed during Fiscal Year 1994 by the Hydraulics Laboratory (HL) of the U.S. Army Engineer Waterways Experiment Station (WES) requested and sponsored by the U.S. Army Engineer District (USAED), Vicksburg.

The report was prepared by personnel of the Waterways and Estuaries Division (WD), HL, and by the Civil Engineering Department of Colorado State University (CSU), Fort Collins, CO.

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The report was prepared under the direct supervision of Messrs. Michael J. Trawle, Chief, Rivers and Streams Branch (RSB), WD; and Thomas J. Pokrefke, Chief, River Engineering Branch, WD; and under the general supervision of Messrs. William A. McAnally, Chief, WD; R. A. Sager, Assistant Director, HL; and Frank A. Herrmann, Director, HL. This report was prepared by Messrs. Pokrefke, Trawle, David L. Derrick, and Billy E. Johnson, and Dr. Nolan K. Raphelt, RSB; and Dr. Chester C. Watson, CSU.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
acres	4,046.873	square meters
cubic feet	0.02831685	cubic meters
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	25.4	millimeters
miles (U.S. statute)	1.609347	kilometers
pounds (force) per square foot	47.88026	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
square miles	2.589998	square kilometers
tons (2,000 pounds, mass)	907.1847	kilograms

1 Introduction

Background

The Demonstration Erosion Control (DEC) Project provides for the development of an overall plan to control sediment, erosion, and flooding in the foothills area of the Yazoo Basin, Mississippi. Structural features used in developing rehabilitation plans for the DEC watersheds include high-drop grade control structures similar to the U.S. Department of Agriculture Soil Conservation Service (SCS) Type C structure; low-drop grade control structures similar to the Agricultural Research Service (ARS) low-drop structure; pipe drop structures; bank stabilization; and a combination of retention and detention reservoirs. In addition, other features such as levees, pumping plants, land treatments, and developing technologies may also be used.

Evaluation of the performance of these erosion control features can contribute to the improvement and development of design guidance for the DEC Project and potentially for similar type projects throughout the United States. Most of the previous Yazoo Basin evaluation has been limited to single-visit data collection, with no comprehensive monitoring of the structures or the effect of the structures on channel stability. The portion of the DEC Monitoring Program being conducted by the U.S. Army Engineer Waterways Experiment Station (WES) is a multiyear program initiated in late Fiscal Year (FY) 1991 and planned through FY 1997; however, to fully document the impacts of the DEC Project will probably require more than 6 years. A monitoring plan for the DEC Project after FY 1997 will be provided at the appropriate time.

Objective

The purpose of monitoring is to evaluate and document watershed response to the implemented DEC Project. Documentation of watershed response to DEC Project features will allow the participating agencies a unique opportunity to determine the effectiveness of existing design guidance for erosion and flood control in small watersheds.

While the objective of previous DEC reports was to document the WES monitoring activities during the period from June 1992 through September 1993 (Raphelt et al. 1993; 1995), the objective of this report is to document the state of the DEC based on the WES monitoring activities.

Approach

To provide the information necessary for the effective evaluation of the DEC Project, the DEC Monitoring Program includes eleven technical areas that address the major physical processes of erosion, sedimentation, and flooding:

a. Stream gauging.

b. Data collection and data management.

c. Hydraulic performance of structures.

d. Channel response.

e. Hydrology.

f. Upland watersheds.

g. Reservoir sedimentation.

h. Environmental aspects.

i. Streambank stability.

j. Design tools.

k. Technology transfer.

The WES portion of the monitoring program has primary responsibility for all technical areas except stream gauging and environmental aspects. The primary responsibility for these technical areas rests with the U.S. Geological Survey (USGS) and ARS, respectively.

Technical Area Descriptions

The following is a general description of the work being performed by WES in the nine technical areas.

Data collection and data management

The purpose of the data collection and data management technical area is to assemble, to the extent possible, all data that have been accumulated to date in the DEC Project, and develop an engineering database that will be periodically updated as new monitoring data are collected and analyzed. The database resides on an Intergraph workstation, and access to the database is made user-friendly with Intergraph software. The database is available to all participants in the monitoring program to provide for analysis and evaluation of the various elements of the DEC Project. In addition to the extensive hydraulic and sedimentation data being collected in the monitoring program, the database contains aerial photography, USGS digital elevation grids, USGS quadrangle maps, and project feature locations and information.

Hydraulic performance of structures

Six grade control structures were selected for detailed data collection to evaluate hydraulic performance. The structures were selected on the basis of special features, including high drop, low drop, significant upstream flow constriction, limited upstream flow constriction, free flow, and submerged flow. The structures were instrumented to collect data to evaluate discharge coefficients, energy dissipation, flow velocity distribution, and effects of submergence on performance. All riprap bank stabilization measures in each watershed will be visually monitored and problem areas identified. A minimum of three riprap bank stabilization installations including riprap blanket revetment, riprap toe protection, and riprap dikes were selected to evaluate toe and end section scour. Data are being collected during runoff events to measure magnitude and location of maximum scour and the corresponding hydraulic parameters. This technical area also included the construction of a physical model of a low-drop structure. The model was used to determine if modifications can be made to the low-drop structure design that either maintain or enhance performance characteristics at a reduction in cost.

Channel response

The channel response monitoring focuses on two major areas: channel sedimentation and channel-forming discharge. Monitoring for channel sedimentation includes an annual geomorphic update of selected watersheds. In addition to the geomorphic update, 23 sites where structures exist or are anticipated were selected for intensive monitoring over the life of the program. Channels upstream and downstream of the selected structures are being monitored for cross-section changes, thalweg changes, berm formation, bank failure, and vegetation development. Five additional sites where no structures are planned are also being monitored. These five sites serve as a control group and assist in the evaluation of channel response to structures. Photographic documentation of structures and channels is being conducted and included in the database. A subset of these structures and channels is being instrumented

for stage, discharge, suspended sediment concentration, and bed-load material measurements. The numerical sediment transport model HEC-6 and the SAM computer program have been used to predict the stability of channels monitored by this work effort. Also, the DEC watersheds are providing data that will be used to test design procedures and techniques for the channel-forming discharge concept. Successful development of such channel-forming discharge methodology could result in significant design cost savings for the DEC Project.

Hydrology

Rainfall provides the energy to sustain erosional processes. The ability to measure rainfall and compute runoff accurately is crucial in the design of stable flood-control channels. Accurate flow rates are needed to design functional project features properly and maintain stability in the channel system as well as monitor the project. CASC2D hydrologic models of a selected number of watersheds have been developed. Hydrologic modeling and hydraulic structures monitoring are being coordinated so that hydrologic parameters used in CASC2D can be determined at locations in the watersheds where USGS gauging stations do not exist.

Upland watersheds

ARS has been given the primary responsibility for this technical area. WES was not active in this area during FY 1994. The two items related to the upland watersheds to be monitored by ARS are system sediment loading (sediment yield) and sediment production from gully formation. Stabilization measures being installed to reduce upland erosion will be monitored by ARS over the next 3 years to determine if a measurable change in the quantity of sediment being transported from watersheds occurs. Data collected by USGS and ARS over the past 7 years will be analyzed and interpreted by ARS to serve as the base for future comparisons. The numerical modeling of sediment runoff from watersheds by WES is planned as part of the analysis and interpretation process. Also, sediment production from two or three active gullies will be analyzed by ARS by comparing surveys made prior to the design of drop pipes and the survey made just prior to construction of the drop pipes.

Reservoir sedimentation

The major sources of reservoir deposition are upland erosion, erosion of the channel banks, and erosion of the channel bed. The reduction of the inflowing sediment load is being addressed in the channel response, streambank stability, and upland watershed technical areas. WES is using the results of the analysis performed in these areas to determine the effects of the project on reservoir sedimentation.

Streambank stability

Streambank stability depends on hydraulic parameters related to flow conditions and the characteristics of the materials in the banks. All channels will be visually monitored periodically to determine reaches that are experiencing severe bank stability problems. In addition to the overall visual monitoring, five sites where aggradation is occurring and five sites where bank caving is occurring were selected for detailed monitoring. At the selected sites, surveys of closely spaced sections will be made semiannually to document changes. After sufficient data have been collected, appropriate numerical models will be applied to determine if existing numerical techniques can be adapted to predict bank stability and/or bank failures accurately.

Design tools

The procedures and techniques used in the design of the different features of the DEC Project have the potential for national and international applications. Effective application of these design procedures and techniques may require development of computer-based packages and the validation of numerical models such as CASC2D, HEC-6, SAM, and BURBANK. In conjunction with ongoing research, WES is developing design tools specifically targeted for the planning and design of stable flood-control projects.

Technology transfer

Technology transfer is an important part of the DEC Project and will be given high priority at WES during the life of the monitoring program. When appropriate, WES personnel present results at national and international technical conferences and symposiums. When appropriate, WES personnel will host workshops and training classes for both Corps and non-Corps personnel. WES will annually report on the DEC monitoring program using several different formats. For FY 1994, these included the following:

a. An updated engineering database on the Intergraph system including aerial photos, surveys (channel and structural), results of numerical studies, etc.

b. A detailed WES technical report on monitoring, data collection, data analysis, and project evaluation.

2 Data Collection and Data Management

The WES data collection effort is in direct support of the other DEC monitoring functions. Data being collected consist of water surface elevations and flow rates obtained from the various streams and rivers in the DEC watersheds. The primary use is as input to hydraulic and hydrologic models. A secondary use is in the analysis of the performance of hydraulic structures.

The raw data are recorded in feet of water relative to an arbitrary reference point. Depending on the type of instrumentation used, the data must be added to or subtracted from a known datum to represent the true water surface elevation. In the case of the flow rate measurements, the data are recorded as velocities associated with known cross-sectional areas. From these, a flow rate is calculated for a given cross section.

The data collection effort for FY 1994 involved the following activities:

a. Continue stage data collection at established gauging locations.

b. Continue discharge measurements at established stream gauging locations.

c. Continue quality control processing of stage data.

d. Develop stage-discharge rating curve for established gauging locations.

e. Develop discharge curves for established gauging locations.

Stage Data Collection

Stage data were gathered at the same locations used in FY 1993 (Raphelt et al. 1995). The purposes of the gauges installed in FY 1994 were to

(a) provide back-up data in case of electronic instrument failure, (b) verify electronic data at high stage events, and (c) provide high stage levels at weirs for better water profile definition.

The number of new crest gauges added in FY 1994 at each site is listed in the following tabulation:

Creek Site	New Crest Gauges
Harland	6
Fannegusha	2
Abiaca	1
Coila	1
Lick	1
Red Banks	2
Lee	1
Hickahala	3
Hotophia	1
East Fork Worsham	1
James Wolf	2

The location numbers given in this and other DEC reports are WES designators solely for the purpose of record maintenance and linking to specific streams.

During FY 1994 stage data collection, failures were due primarily to three causes. The first, and largest, cause was electronic instrument failure, primarily of the pressure sensing gauges. These failures were a continuation of the problems experienced in FY 1993. The problems were seen in all three parts of the gauge system, i.e., transducer, logger box, and data card. A review of the equipment, application, and installation was made which resulted in developing an improved method of using this type of gauge. The new method was used at all locations in FY 1995.

The second cause of data gathering failures was due to construction at or near the instrument location site. A weir was constructed at Fannegusha Creek below the instrument sites which caused water flow to be abnormal. Lick Creek had a high-drop structure constructed in the middle of the instrument site which altered the flow significantly. Hickahala Creek had a county bridge replaced immediately upstream of the instrument locations.

Vandalism was the third major cause of failure in data gathering. Instruments were shot out at Red Banks Creek, East Fork of Worsham Creek, and the Middle Fork of Worsham Creek. A family living in the Worsham Creek area was contacted concerning the shootings. They offered to help by

contacting the sheriff when shots are fired. Vandalism in that area seems to be declining. Instrument wires were cut at Otoucalofa Creek, and unsuccessful attempts at vandalism were observed at Harland Creek, Sarter Creek, and Abiaca Creek.

Discharge Measurements

Stream gauging for discharge data continued at all scheduled sites by WES and USGS personnel. Results have not been as good as planned. The two primary factors contributing to the difficulty are, first, the inability to accurately forecast the quantity and time of rainfall for a geographically small watershed. Secondly, the short duration of an event has made "catching" an event quite difficult. These two factors are exacerbated by the distance to be traveled by WES and USGS personnel to the site location. Alternative methods of gathering this information were in place in FY 1995.

Stage-Discharge Curves

The available stage-discharge information in conjunction with theoretical calculations have enabled stage-discharge rating curves to be created for Hickahala Creek; Hotophia Creek; the East, Middle, and West Forks of Worsham Creek; Burney Branch Creek; James Wolf Creek; and Long Creek. The rating curves have been used to create discharge hydrographs for stages measured in FY 1993 and FY 1994. The rating curves and subsequent discharge hydrographs may require some future adjustment as more field data are gathered.

Figure 1 is a typical example of the stage data from Long Creek that are available on the DEC streams from the monitoring effort. Figure 2 is the rating curve developed for Long Creek using theoretical calculations. Similar curves have been developed on streams using USGS discharge measurements collected over the last few years. Using the stage hydrographs and the developed rating curves, a discharge hydrograph can be developed for the particular stream. Figure 3 is an example of such a discharge hydrograph. For clarity the examples presented show data for a 90-day period of stage and corresponding discharge. Table 1 provides a list of the monitored DEC streams that are available in the U.S. Army Corps of Engineers hydrologic data processing program called DSS (Data Storage System). As stated previously, the rating curves developed for the DEC streams were based on theoretical computations or USGS measurements. In the rating curve column of Table 1, the "US" means that the curve was developed using USGS measurement, and the "TH" means that the curve was developed using theoretical computations.

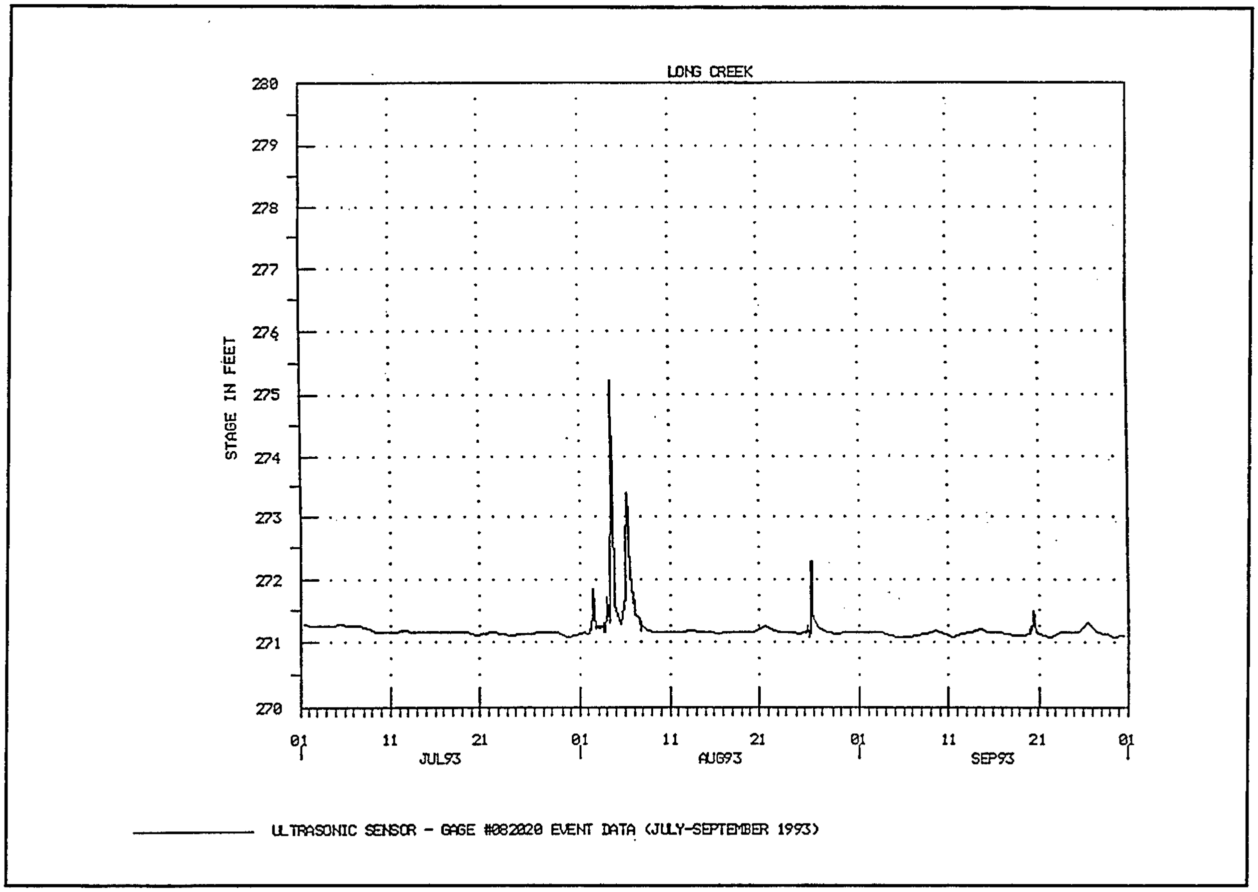

Figure 1. Example of stage data

In the same column the "94" and "95" indicate the fiscal year that the curve was developed.

Engineering Database/Geographical Information System

The purpose of the engineering database/Geographic Information System (GIS) is to serve as a repository for all design, analysis, and monitoring data collected on the DEC Project. The engineering database/GIS concept was chosen for the DEC Project because it allows for the storage, retrieval, analysis, and graphical display of all data. When completed, it is anticipated that the database will contain design data for all project features such as low- and high-drop structures, bank stabilization structures, floodwater-retarding structures, channel improvements, levees, riser pipes, and box culverts. Every effort was made to include data from all participating agencies in the DEC Project.

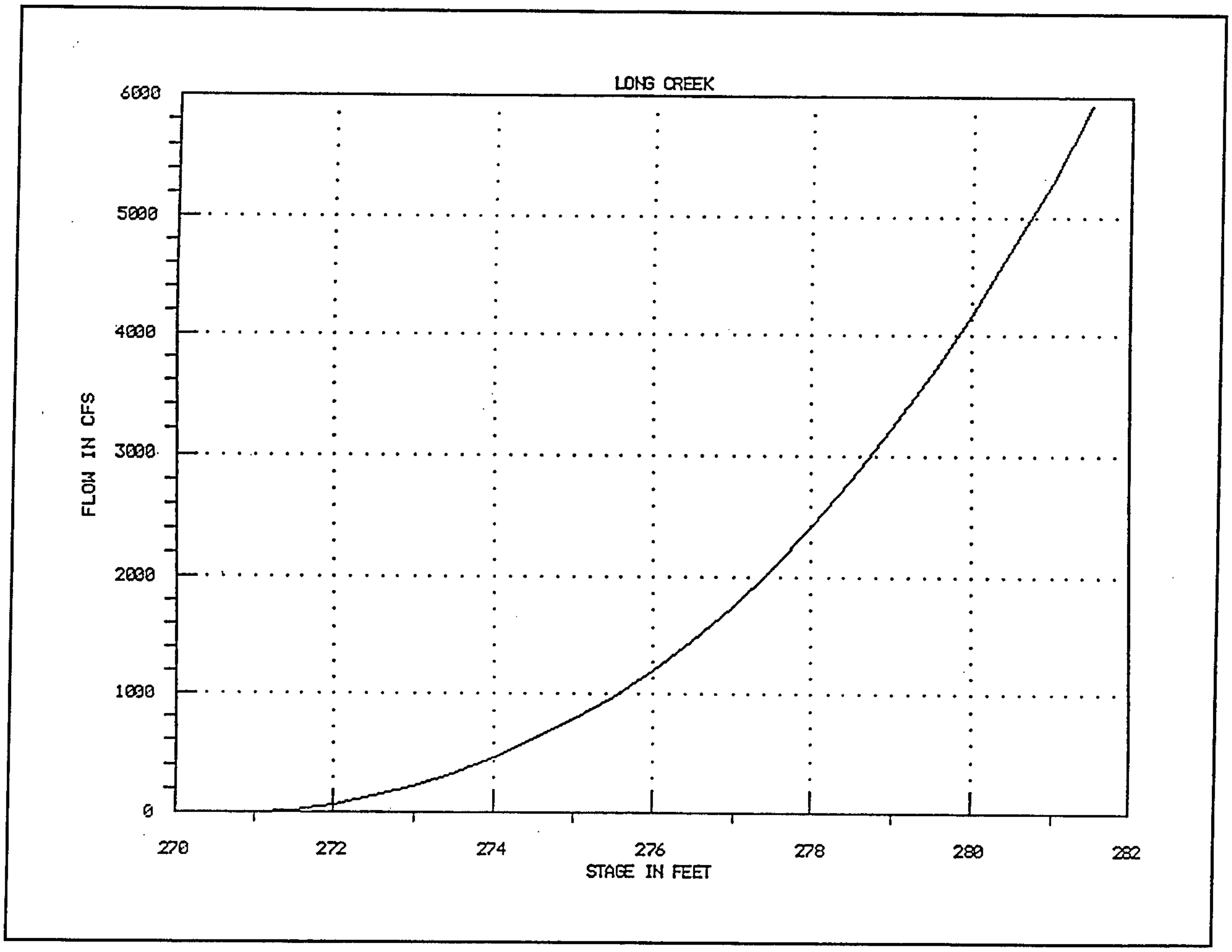

Figure 2. Example of rating curve

The database will contain an index of all studies, analyses, and published reports for the DEC Project. Significant reports from the index list will be incorporated as documents into the database. The database will be tied to the GIS system for graphical display of the data. The Informix relational database is being used to store the data, which allows analysis of project features. In addition to the Informix relational database, the U.S. Army Engineer Hydrologic Engineering Center (HEC) data storage system, HECDSS, will be embedded in the engineering database/GIS. The HECDSS database will contain stage, discharge, and cross-section data and will serve as a base for running numerical models. It is anticipated that HEC-1, HEC-2, and later in the project, two-dimensional hydrology and three-dimensional hydraulic models will run from data stored in the database.

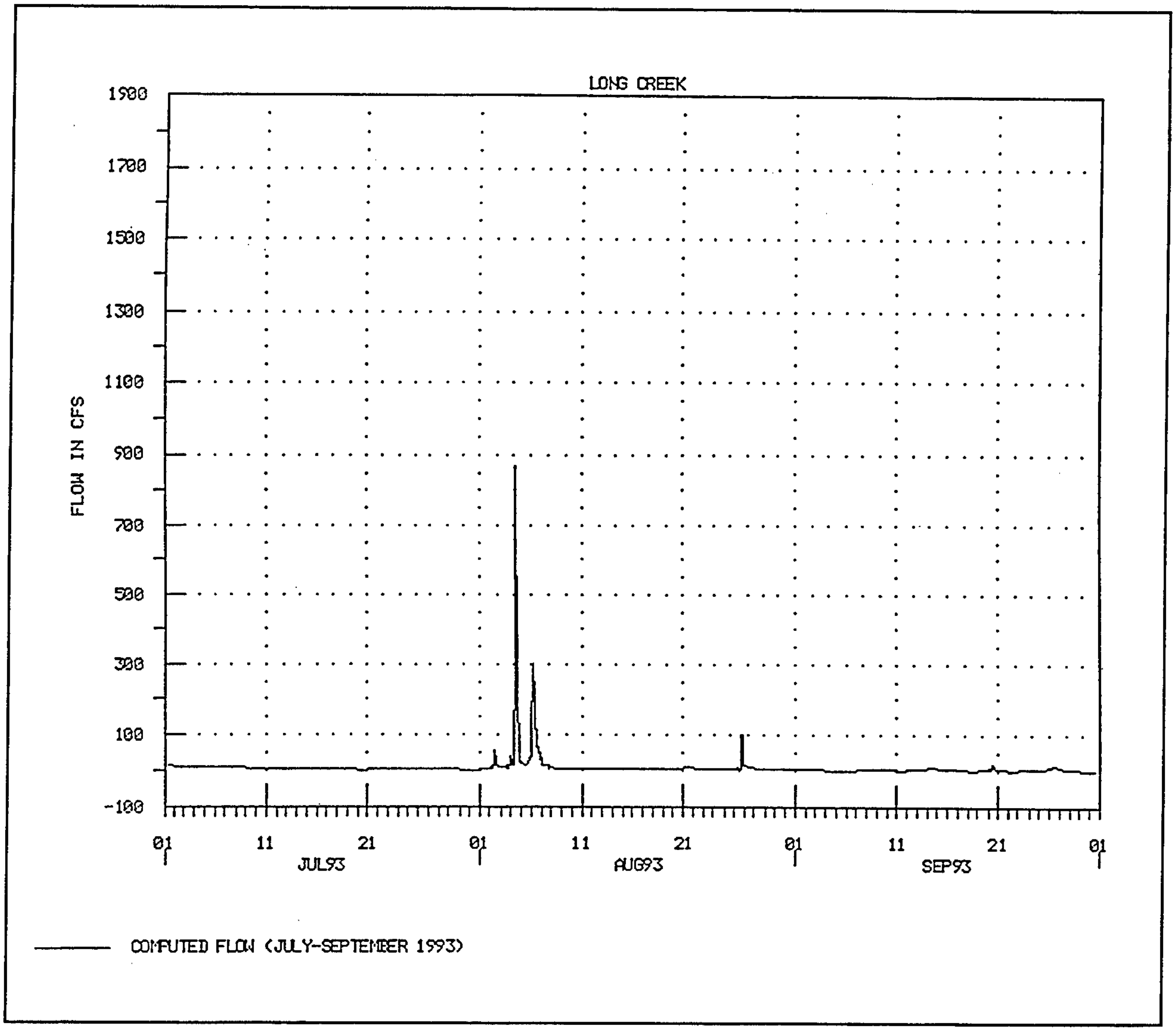

Figure 3. Example of discharge hydrograph

Computer Hardware and Software

The engineering database/GIS is being developed on the Intergraph 6040 workstation. The engineering database/GIS uses a number of MGE products. MGE is the umbrella under which Intergraph's GIS and database management software run. The system uses the Microstation software package. Microstation capabilities include computer-aided drafting and design (CADD), editing and placement of project features, editing and drawing of project features, and design and development of new design files. Also under MGE are Imager for image processing, IVEC for vectorization of scanned data, and Grid Analysis. Imager is used for image processing and with Grid Analysis for the hydrologic studies. Grid Analysis is used to develop grids for soil type, land use, slope, and elevation. MGE Terrain Modeler and a number of MGE

translator programs translate Digital Line Graph (DLG) and Digital Elevation Model (DEM) data into the Intergraph format. It is anticipated that two additional Intergraph pieces of software will become important in the database. The DBS software will be used for document storage and retrieval; and the Inroads program will be used to store terrain model data and survey data, develop HEC decks for two- and three-dimensional models, and monitor surveys and changes in cross sections and survey areas. The HEC database will be used to store stage, discharge, and cross-section data.

Status

As of 30 September 1994, the database contained SCS curve numbers on a 1-acre[1] grid for 13 of the DEC watersheds (Abiaca, Batupan Bogue, Black-Fannegusha, Burney Branch, Cane-Mussacuna, Coldwater, Hickahala-Senatobia, Hotophia, Hurricane-Wolf, Long, Otoucalofa, Pelucia, and Toby Tubby). The 1:24,000 digital quadrangle maps and DEM's have been incorporated into the engineering database for all of the DEC watersheds. Initially, streams and roads from the 1:100,000 USGS DLG's were incorporated into the database. As the 1:24,000 DLG data become available, they will be added to the database. Spot-View satellite photography has been incorporated into the database and is used as a visual reference for all project features. Satellite photography at 10-m resolution is in the database for all DEC watersheds. The engineering database consists of the locations and design parameters for all existing structures constructed by the U.S. Army Engineer District, Vicksburg, for riser pipe, low-drop, and high-drop structures; bank stabilization; and box-culvert grade control structures. Locations of proposed and constructed levees, floodwater-retarding structures, and channel improvement and box control structures are also in the database. Land use data for Abiaca, Cane-Mussacuna, Coldwater, Hickahala-Senatobia, Hurricane-Wolf, and Long Creek watersheds are in the database on a 1-acre grid. Soil type data for Abiaca, Batupan Bogue, Black-Fannegusha, Burney Branch, Cane-Mussacuna, Coldwater, Hickahala-Senatobia, Hotophia, Hurricane-Wolf, Long, Otoucalofa, Pelucia, and Toby Tubby Creek watersheds are in the database on a 1-acre grid. Elevation and slope data for Abiaca, Batupan Bogue, Black-Fannegusha, Burney Branch, Cane-Mussacuna, Coldwater, Hickahala-Senatobia, Hotophia, Hurricane-Wolf, Long, Otoucalofa, Pelucia, and Toby Tubby Creek watersheds are in the database on a 30-m grid. The database contains all major tributaries and highways for the 15 DEC watersheds. The 1:100,000 digital DLG files are the source of the stream and highway data. A summary of the data contained in the database at the end of FY 1994 is given in Table 2.

[1] A table of factors for converting non-SI units of measure to SI units is found on page vi.

3 Channel Response

Introduction

The 23 selected sites being monitored in the program include approximately 15 existing low-drop structures, 4 existing high-drop structures, a channelization project, chevron dikes, riprap and bioengineered bank stabilization, a sediment basin, and 6 control reaches in approximately 34 miles of study reach. These sites have been selected to represent many of the different DEC watersheds, types of channel planform and sediment gradation, particular causes of instability, types of channel rehabilitation, and locations of special interest.

Each monitored site was evaluated by Colorado State University (CSU) under contract as part of the DEC monitoring program. This included an analysis using both the BURBANK and SAM programs. In the incised channels of the Yazoo Basin, a fixed lateral boundary for the channel is a simplification because considerable channel widening takes place as a result of degradation. Bank failures provide considerable sediment inflow to the channels. Since this adjustment should be considered in developing models for analysis, a modification of previous bank stability models was made by CSU to develop the BURBANK computer program. BURBANK is a basic language computer program that quickly assesses the bank stability for a channel reach. The program uses a HEC-2 data input file to describe channel morphology, and the user inputs the friction angle, specific weight, and cohesion of the bank materials. An important assumption is that the bank materials are homogeneous. Output from the program is the percentage of the bank at risk of failure for existing or for user-supplied amounts of degradation. The program can also create a new HEC-2 file that approximates the post-failure and clean-out condition. For a detailed description on the BURBANK program see Burgi, Watson, and Gessler (1995).

The WES SAM model is a flexible program for the computation of sediment transport at a section, sediment yield, channel roughness, and related computations (Thomas et al., in preparation). The SAM program couples resistance and sediment transport equations to solve for the channel dimensions of width, depth, and slope. A family of solutions for width and slope is computed that

describes a series of width and slope combinations that provides for water and sediment continuity for a cross section. SAM also provides for combining several cross sections in a reach to provide a reach-averaged condition. Table 3 summarizes the discharge, width, depth, slope and sediment size for each of the sites.

Monitored Sites

Each site is discussed in this section. For each site the location, a site map, a thalweg profile, and summary table of BURBANK and SAM analyses are presented. In the summary tables of BURBANK and SAM analyses (Tables 4 through 27) the upper portion of the table depicts the BURBANK analyses of bank stability for the channel. The first line states the unit weight and cohesion values selected to be representative of the site. Next are three columns of results for percent of bank at risk of failure for each of three monitoring years (1992, 1993, and 1994), and for conditions of friction angle values of zero and a selected representative value. The right of the table lists results of percent of bank at risk for a friction angle of zero and for additional degradation of 1, 2, or 3 ft. The lower portion of the table presents the percentages of width and slope of the minimum stream power cross section for the equilibrium transport of 1,000 mg/ℓ that the existing channel has attained. This is computed for each of the three surveys — 1992, 1993, and 1994. For example, on Abiaca Creek, Site 3, the SAM solution of minimum stream power at a transport rate of 1,000 mg/ℓ for the width is 90 ft and for the slope is 0.000665 ft/ft. In 1994, the actual width was surveyed at 70 percent of the width (63 ft) and the hydraulic energy gradient was computed using HEC-2 to be 160.9 percent of the slope (0.00106 ft/ft). This means that the channel is transporting more than 1,000 mg/ℓ sediment concentration, and that the channel will widen to achieve minimum stream power.

The sites presented are grouped by stream for the watersheds on which more than one site exists. Site numbers were originally assigned in order of selection; however, for convenience in use, the sites are presented in alphabetical order by stream name or watershed.

Abiaca Creek Watershed, Sites 3, 4, 5, 6, and 21

Five sites have been selected in the Abiaca Creek watershed. These sites can be found on the Seven Pines quadrangle map. The drainage area of the watershed is about 100 square miles and SCS reservoirs control approximately 60 percent of the watershed. Coila Creek is the principal tributary to Abiaca Creek, and the Coila Creek watershed is approximately 76 percent controlled. Upstream of the confluence of Abiaca and Coila Creeks, Abiaca Creek is about 49 percent controlled. Along with the importance of this watershed supplying water to a downstream wildlife area, this watershed has been severely affected by sand and gravel mining.

Site 3, shown in Figure 4, is located in T17N, R3E, Section 20, at the Highway 17 crossing of Abiaca Creek. The approximate watershed area at this site is 26.5 square miles. This site was selected because of the relative stability of the channel at this location, and the fact that it is upstream of the gravel mining location. The streambed at Site 3 is composed primarily of sand with minor amounts of gravel. The banks are generally well-vegetated with mature vegetation down to the low-water surface; however, erosion of the outside bank of the bendways was noted. There is notable bank erosion on the upstream portion of this site extending upstream of Highway 17 where a bend is actively migrating. This is a local erosion situation, with no indication of overall system instability. Bioengineering methods could be used to stabilize these sites. The thalweg profiles (Figure 5) indicate that local variation, primarily scour, has occurred on the order of 2 to 3 ft vertically.

Site 4 is on Abiaca Creek and extends approximately 4,000 ft upstream from the confluence of Coila Creek as shown in Figure 6. This site is located in T17N, R2E, Section 4, and has a watershed area of approximately 44 square miles. Site 4 is located approximately 1.8 miles downstream of a major sand and gravel processing operation that can be associated with an increased supply of suspended and bed material load. Streambanks in this reach are relatively stable and the bed gives the appearance of an aggraded reach. The most significant change in this watershed has occurred at the confluence of Abiaca and Coila Creeks, indicating aggradation since 1992 (Figure 7). During the October 1994 field inspection, Abiaca Creek flow was a milky, high suspended sediment load flowing into a relatively clear Coila Creek flow. The island immediately downstream of the confluence had shifted to a bar attached to the right bank, with all flow along the left bank. The downstream 500 ft of Coila Creek is actively eroding and the confluence may shift downstream by next year.

Site 5 is actually located on Coila Creek, but is in the Abiaca Creek watershed. The site extends upstream approximately 4,000 ft from the confluence with Abiaca Creek as shown in Figure 8 in T17N, R2E, Section 4. The site has a watershed area of approximately 42 square miles, very similar to Site 4, which allows the comparison of two almost equal size drainage basins. SCS reservoirs control a high proportion of the Coila Creek basin, and the gravel mines on Coila Creek are not as active as those along Abiaca Creek. The thalweg profile indicated that some aggradation has occurred on the downstream portion of this site with little or no changes in bed elevations for the remaining portion (Figure 9).

Site 6 is located on Abiaca Creek where the stream emerges from the hill line into the flatter Yazoo Delta in T17N, R1E, Sections 13 and 14, as shown in Figure 10. The drainage area at this location is approximately 99 square miles. This is also the site of the Pine Bluff gauging station with records from 1963 to 1980. This station has been reactivated and includes a

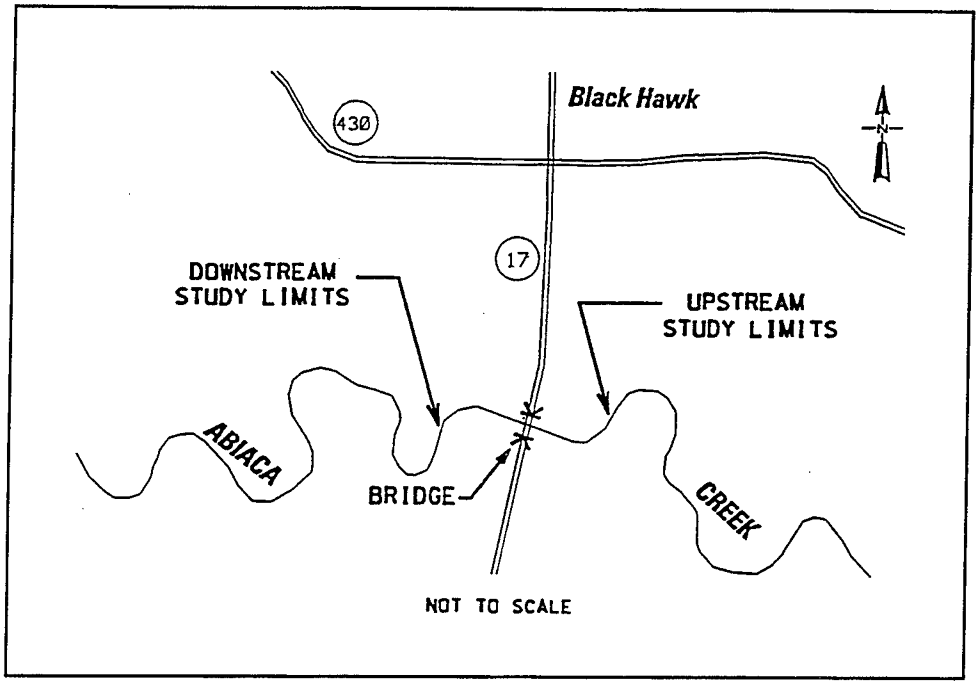

Figure 4. Abiaca Creek, Site 3

pumped sediment sampler. The study reach extends approximately 4,000 ft downstream of the Pine Bluff gauging station. The thalweg profiles (Figure 11) indicate that local variation, primarily scour, has occurred on the order of 2 to 3 ft vertically.

Site 21 is near the mouth of Abiaca Creek at Highway 49 as the stream enters the wildlife area, Section 18, T17N, R1E. The Vicksburg District has designed a sediment trap basin for this location by setting the levees back and allowing frequent overflow of the stream. The reach is approximately 4,000 ft in length and is shown in Figure 12. The thalweg profiles (Figure 13) indicate that local variation, primarily deposition, has occurred on the order of 2 to 3 ft vertically; however, it should be noted that surveys were taken on this site only in 1993 and 1994.

No significant bank erosion was indicated using the BURBANK program. As shown in Tables 4 through 8, only Site 3 has a steeper slope than the 1,000-mg/ℓ minimum slope used as a basis for comparison, indicating that the sediment load is greater than 1,000 mg/ℓ at the site.

Channelization of the lower basin during the early 1920's set in motion a complex cycle of channel incision and widening. Continued mining of sand and gravel in the watershed complicates rehabilitation of the watershed. A sediment trap designed by the Vicksburg District to be located immediately

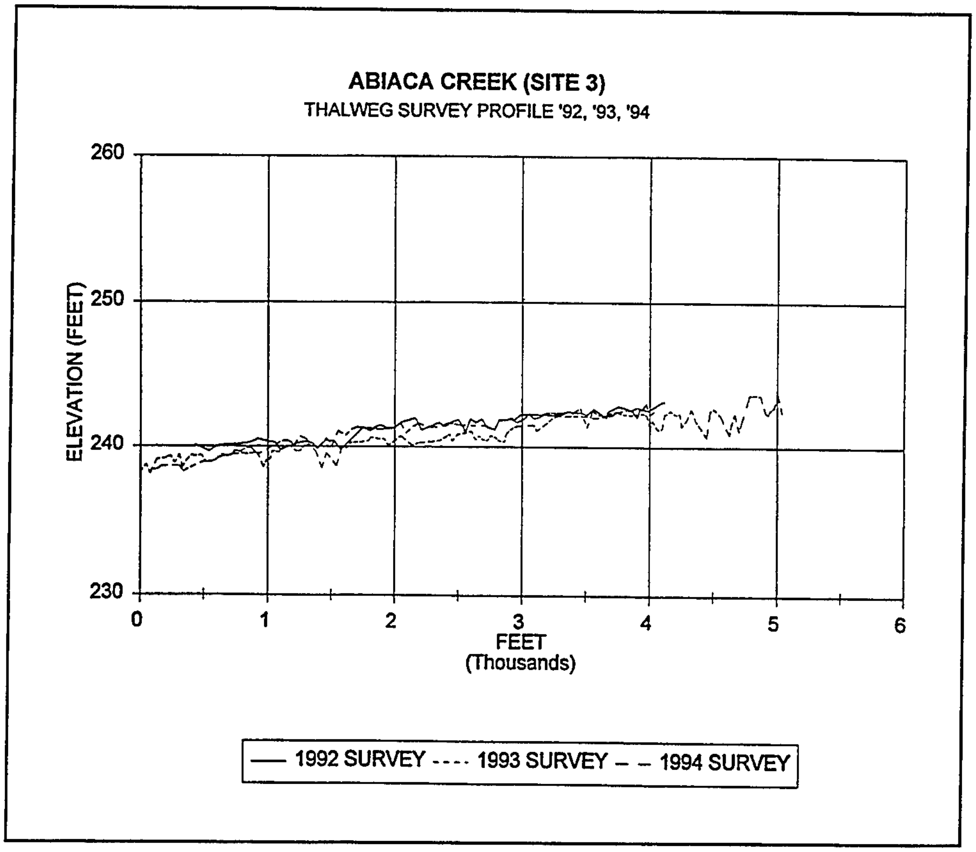

Figure 5. Abiaca Creek, Site 3, thalweg profile

upstream of the wildlife area had not been constructed as of October 1994. The complexity and importance of the watershed emphasize the purpose of these study sites.

Burney Branch, Site 12

Site 12 (Figure 14) is located on Burney Branch near Oxford, MS. The study reach begins at the Highway 7 crossing of Burney Branch and extends downstream for a distance of approximately one mile through a reach containing two SCS high-drop structures. Burney Branch has a drainage area of approximately 10 square miles at this location. The site can be located on the Oxford quadrangle map, T9S, R3W, Sections 4 and 9.

The two high-drop structures have been very successful in rehabilitating this reach of Burney Branch. Both structures were constructed in 1982. These structures were designed to contain the 100-year discharge and include the

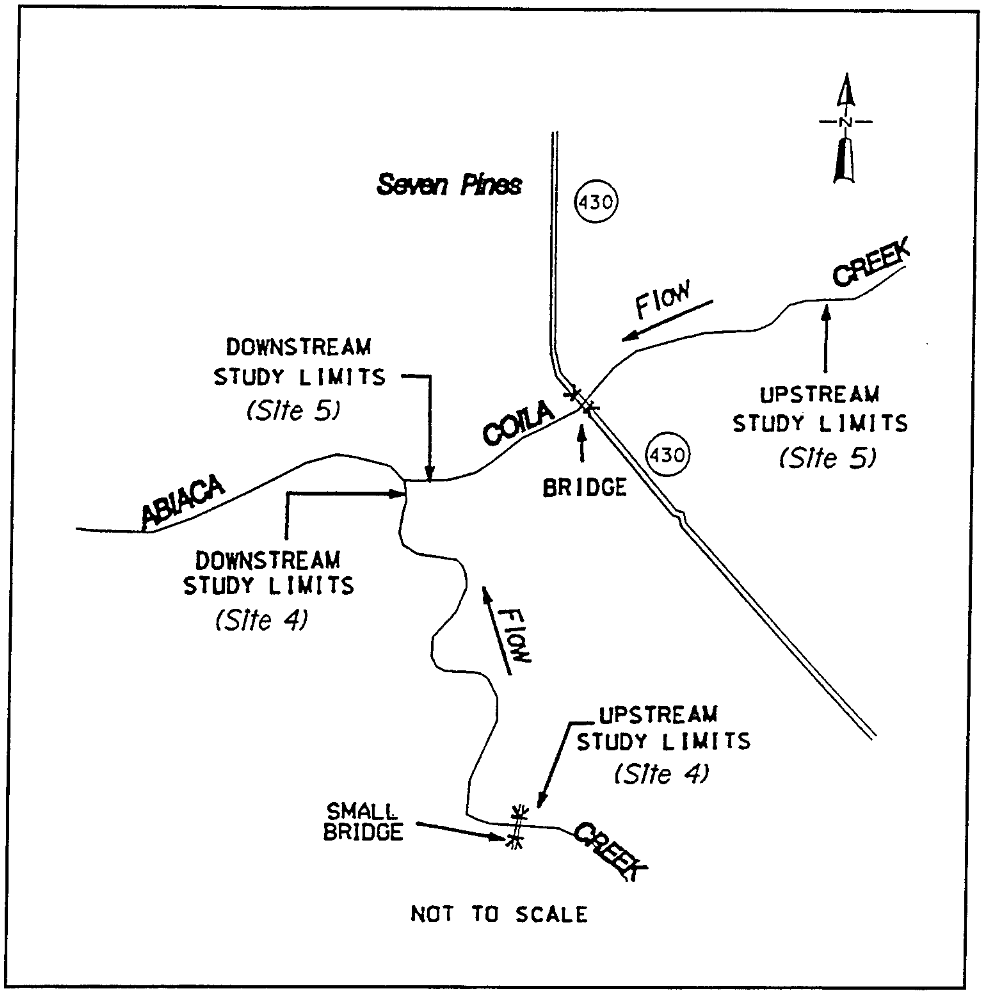

Figure 6. Abiaca Creek, Site 4

provision for floodplain storage using valley dams in conjunction with each structure. The original design of the structures provided for a bed slope of 0.0008 ft/ft between structures, based on Lane's tractive stress analysis. The 1984 surveyed bed slope was 0.0012 ft/ft, indicating that the upstream sediment yield was greater than planned. Since 1984, several major channel stabilization projects have been constructed upstream. The survey made in January 1992 documented the effects of changes since 1984 and provides data with which to evaluate channel change as sediment supply is reduced. Channel stabilization under conditions of decreasing sediment supply is a situation that will be faced as the success of the DEC programs is realized. Potentially, upstream stabilization can cause stability problems downstream.

As shown in the accompanying thalweg profile (Figure 15), Burney Branch is relatively stable with only minor fluctuations in the bed profile. The two high-drop structures have a significant impact on the reach. BURBANK analyses indicate that only a minor portion of the lower reach is at risk of failure. SAM analyses indicate that the segments at this site are within

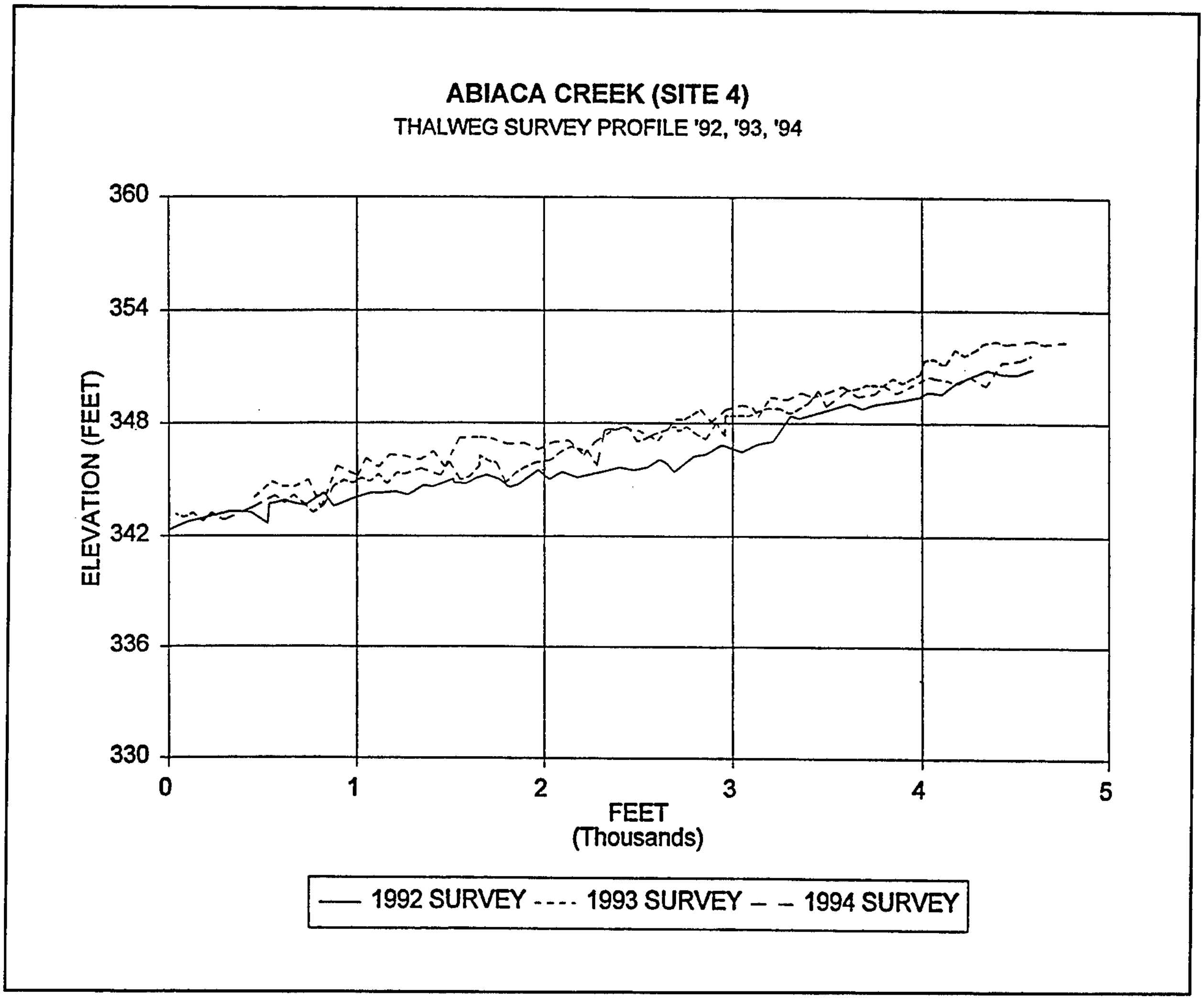

Figure 7. Abiaca Creek, Site 4, thalweg profile

7 percent in width and depth of the minimum stream power channel for transport of 1,000 mg/ℓ (Table 9). The Burney Branch site is an excellent example of channel stabilization, and demonstrates the utility of SAM minimum stream power computations in assessing stability.

Fannegusha Creek, Site 2

Site 2 is located on Fannegusha Creek, in the Black Creek watershed and can be found on the Coila quadrangle map in T16N, R3E, Sections 1 and 2. The watershed area at the site is approximately 18 square miles. This reach was chosen as representing a very unstable sand bed channel. As shown in Figure 16, the study reach is approximately 4,000 ft in length, 2,000 ft upstream and downstream of a county road bridge. A low-drop structure was constructed in 1993 approximately 1,500 ft downstream of the bridge. The streambed appears to be aggrading for a distance of approximately 1,500 to

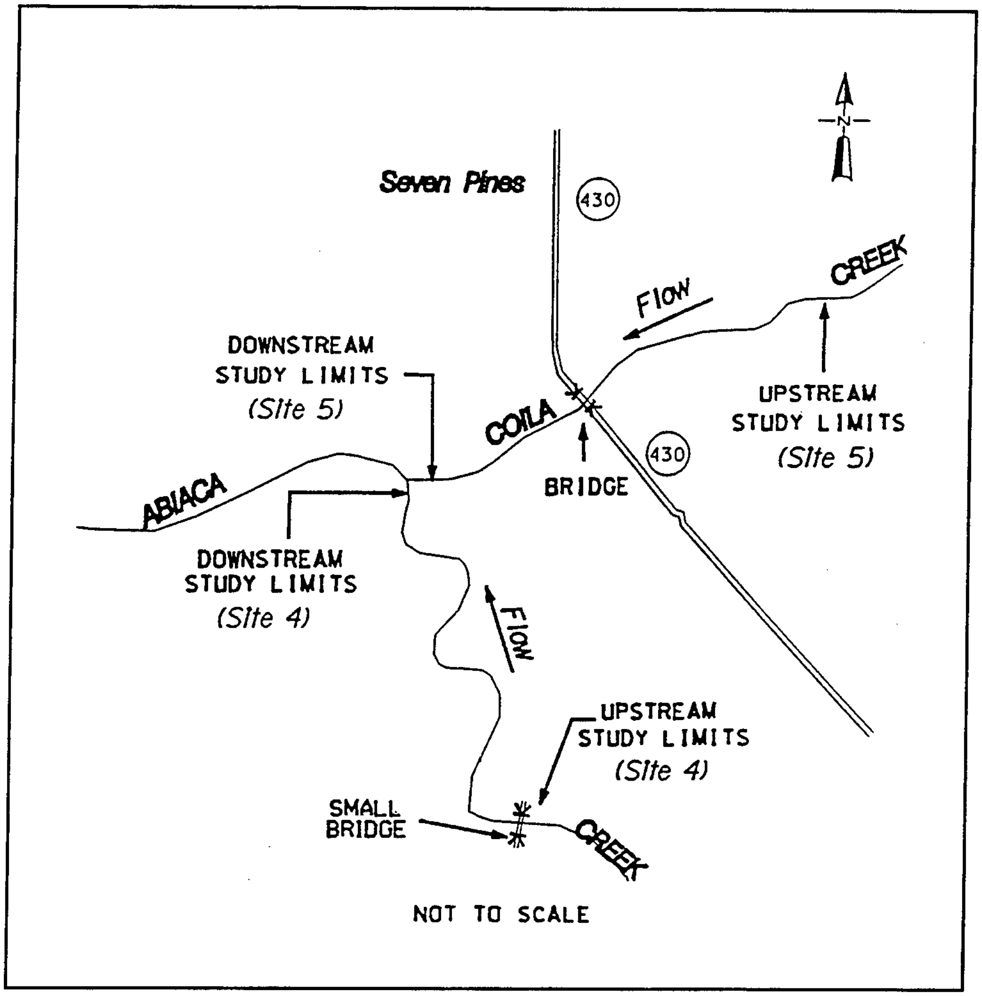

Figure 8. Coila Creek, Site 5

2,000 ft upstream of the structure. The bridge was replaced in 1994 due to channel widening. Observations indicate that the channel will continue to widen as over-steepened banks, due to previous bed degradation, continue to fail. Bed degradation upstream of the bridge will continue as a headcut progresses upstream. At present, the headcut is located approximately 1,200 ft upstream of the bridge.

As shown on the accompanying thalweg profile (Figure 17), the Fannegusha Creek study reach is very active. The lower 1,000 ft of the reach degraded between 1992 and 1993. An ARS low-drop structure was constructed between the 1993 and 1994 surveys approximately at station 10+00. The 1994 thalweg profile indicates that degradation has occurred immediately downstream of the structure, and that a significant amount of aggradation has occurred from about 300 ft upstream of the structure to approximate station 33+00. Therefore, the new structure seems to be functioning well; however, continued surveys will better define the extent of degradation upstream and downstream of the structure. The lower 600 ft of the reach is in very resistant clay, and old,

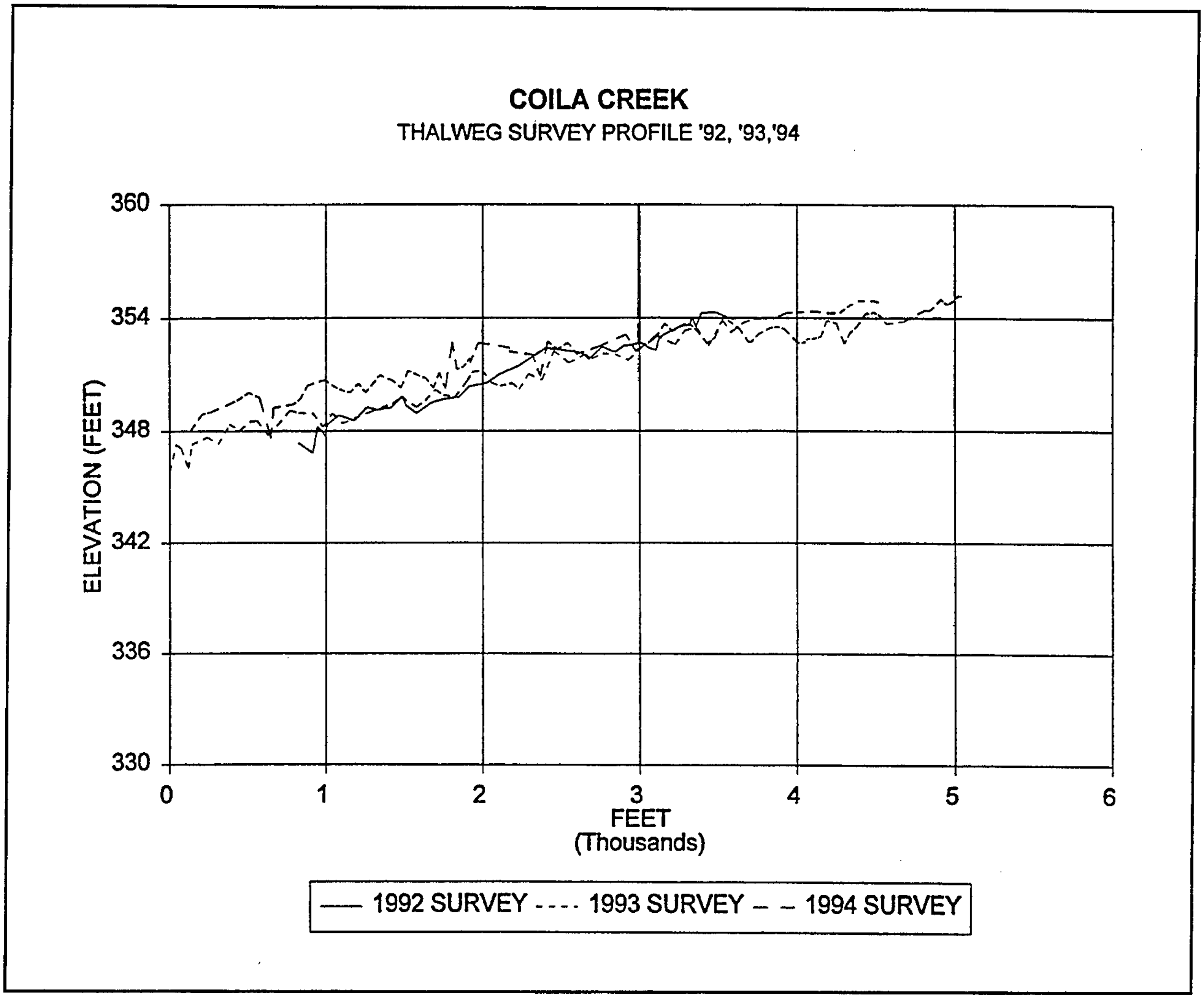

Figure 9. Coila Creek, Site 5, thalweg profile

rotational failures were observed along top bank. These failures gave no appearance of recent movement.

The bridge located at approximate station 20+00 has been rebuilt and the upstream debris pile has been removed. Most of the aggraded reach is being colonized by grasses and young willows, except the left bank immediately upstream of the bridge, where kudzu growth is heavy.

Two headcuts were observed at approximate stations 35+00 and 37+00. Field evidence indicates that the downstream headcut may have moved about 125 ft during the last few years. These features are upstream of the low-water influence of the new structure, and consideration should be given to adding bed and hydraulic control for these sites.

Bank stability analysis indicated little risk of bank failure. The 2-year water surface slope of the reach is 146 percent, and the average stream width is

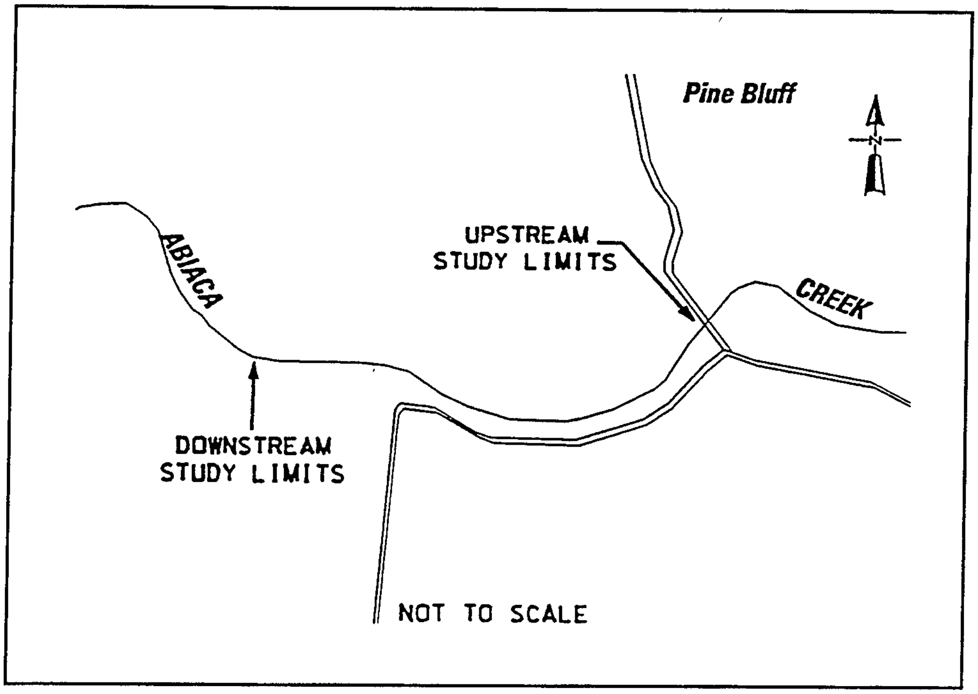

Figure 10. Abiaca Creek, Site 6

144 percent of the slope and width required at minimum stream power for transport of 1,000 mg/ℓ (Table 10). These dimensions indicate that the reach slope has improved in comparison to previous years and that some widening of the average section has occurred, both a result of the new structure. The upstream slope will continue to degrade and flatten as the headcuts migrate upstream.

Harland Creek (upper site), Site 1

Site 1 is located on Harland Creek in the Black Creek watershed. The site is near Eulogy, MS, and can be found on the Lexington quadrangle map in T14N, R1E, Section 22 and 27. The watershed area at the site is approximately 27 square miles. The study reach is approximately 4,000 ft in length, 2,000 ft upstream and downstream of the county road bridge. Harland Creek is a mixed sand and gravel bed stream, exhibiting some of the original meandering tendency shown on the map (Figure 18). The stream is unstable, with bank erosion and significant channel widening. Several areas of massive bank failures have been identified, and these failure sites, along with bed and bank erosion, provide a high sediment yield to the downstream.

The site was chosen because of the mixed bed load, the existence of surveys made before and after riprap stabilization measures were constructed in the reach, and a major reservoir planned immediately upstream of the site. Portions of the study reach were surveyed during 1991 for construction of bank stabilization construction planning. The 1992 field data provide a baseline of

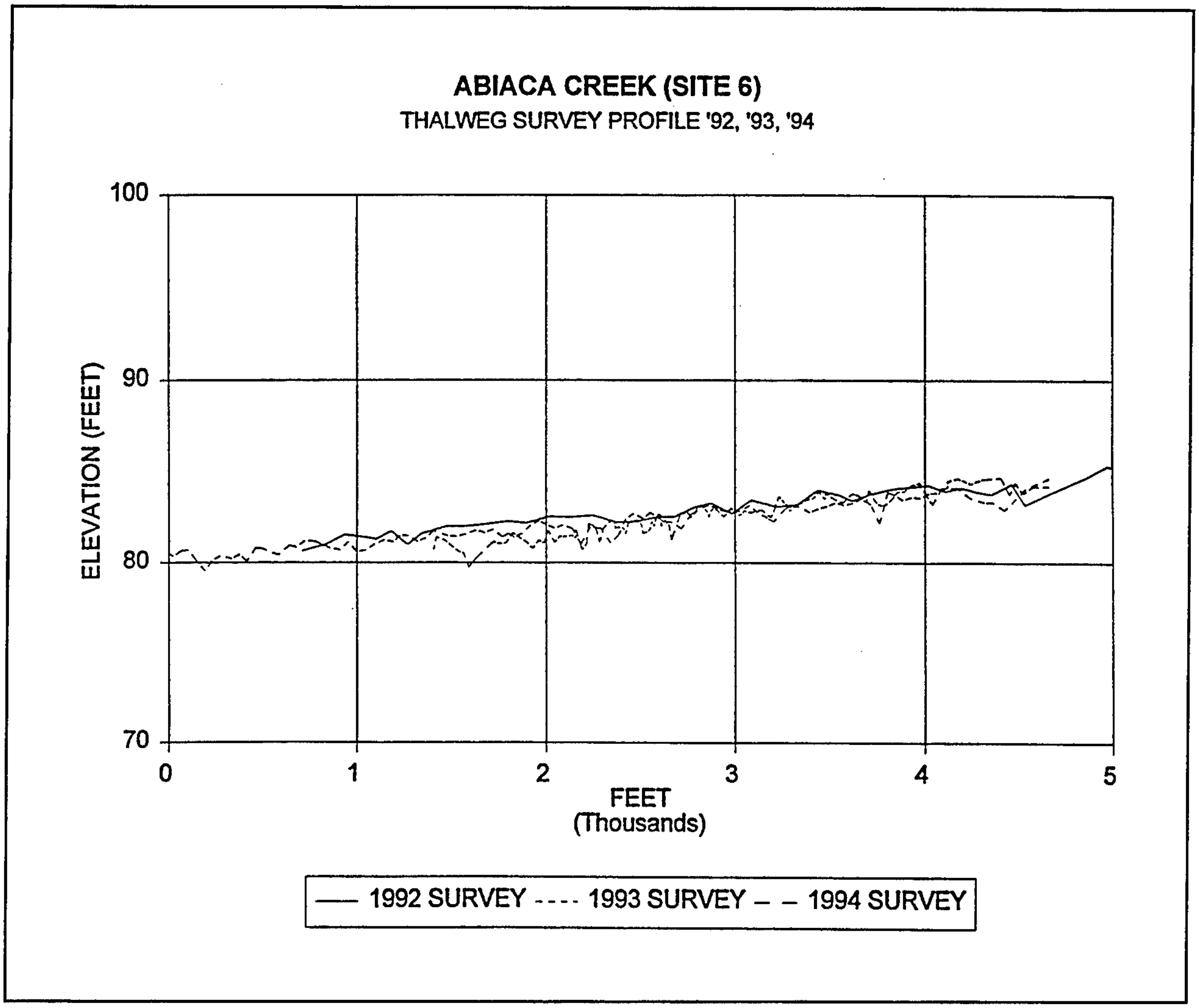

Figure 11. Abiaca Creek, Site 6, thalweg profile

field information for comparison with the 1993 and 1994 surveys, which were made after the channel stabilization was constructed.

As shown in the thalweg profile comparison (Figure 19), the thalweg profile has fluctuated somewhat during 1992, 1993, and 1994. However, no significant changes in a system context are evident. In general, some slight deepening can be noted for 1994, and field evidence indicates that the riprap that was placed after the 1992 survey has resulted in greater water depth. After the June 1994 thalweg survey, thunderstorms during July through October 1994 resulted in severe bank erosion in the unprotected portion of the reach. Field inspection during October 1994 documented severe erosion on the right bank upstream and downstream of the longitudinal riprap at the bridge. Also noted in that field inspection was the low-water filling of the deep pools that had been observed immediately following the riprap placement.

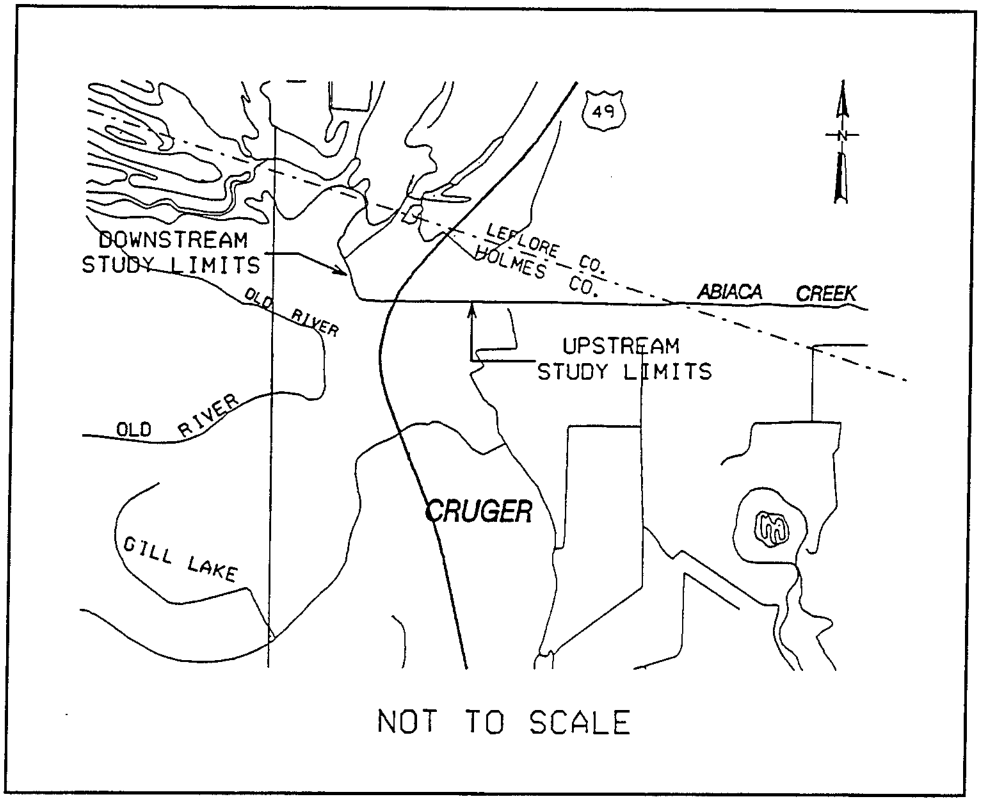

Figure 12. Abiaca Creek, Site 21

Erosion along the outside of the bends upstream of the bridge was documented for a distance of 1,000 ft. Two bioengineered stabilization sites are planned for the right bank upstream of the bridge. A third site is planned along the right bank downstream of the longitudinal riprap that begins upstream of the bridge. A fourth site is planned for the left bank downstream of the left bank tributary at the gap in the left bank longitudinal toe. The downstream tieback at this site has been eroded and riprap is launching. See the section on proposed bioengineering for DEC in Chapter 6.

Bank stability calculations indicate that the surveyed cross sections are stable (Table 11). The 2-year water surface slope of the reach is 81 percent, and the average stream width is 506 percent of the slope and width required at minimum stream power for transport of 1,000 mg/ℓ. These dimensions generally indicate that the bank erosion is due to local hydraulic forces and that channel degradation should not be occurring unless the upstream sediment supply is significantly reduced.

Harland Creek (lower site, willow post), Site 23

From the previous site, the next county road bridge downstream is near the upstream extent of the portion of Harland Creek where willow posts have been

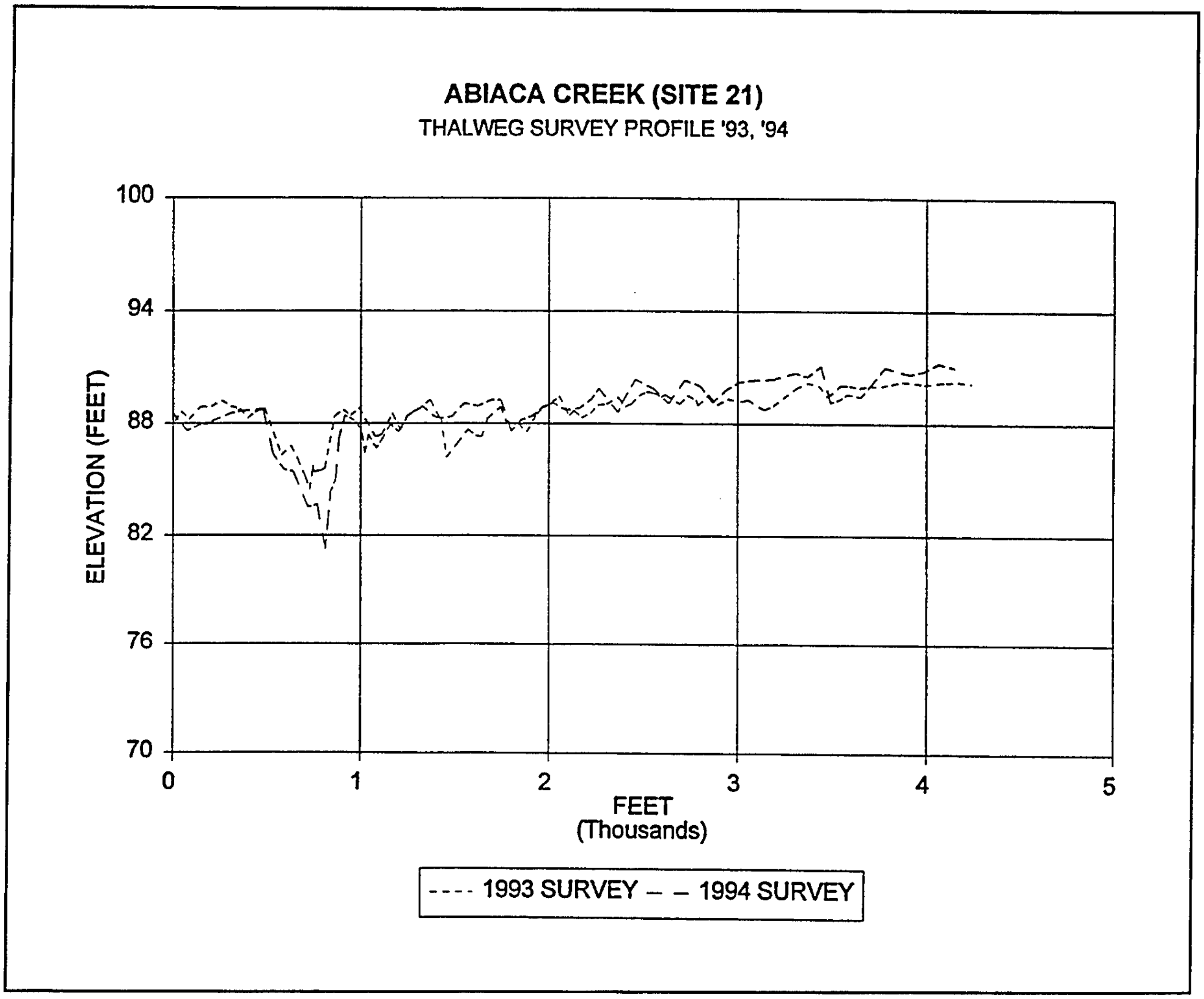

Figure 13. Abiaca Creek, Site 21, thalweg profile

installed by the Vicksburg District. The site continues downstream for approximately 2 miles to the next county road bridge and encompasses an intensive bank stabilization treatment of willow posts and bendway weirs. The site is located in T14N, R1E, Section 11 (Figure 20).

As shown in the thalweg profile (Figure 21), the primary difference between the 1993 and 1994 profiles is the aggradation that has occurred from about station 35+00 upstream to approximate station 75+00. Large gravel bars were observed in the field inspection of October 1994. Some of these bars were in unusual positions for a meandering stream, indicating that the deposits occurred during the recession of a major flood event. It is expected that lower flows will continue to rework these deposits.

Bank stability calculations indicate that the surveyed cross sections are generally stable, with only 2 percent of the 1994 bank at risk (Table 12). The 2-year water surface slope of the reach is 116 percent, and the average stream

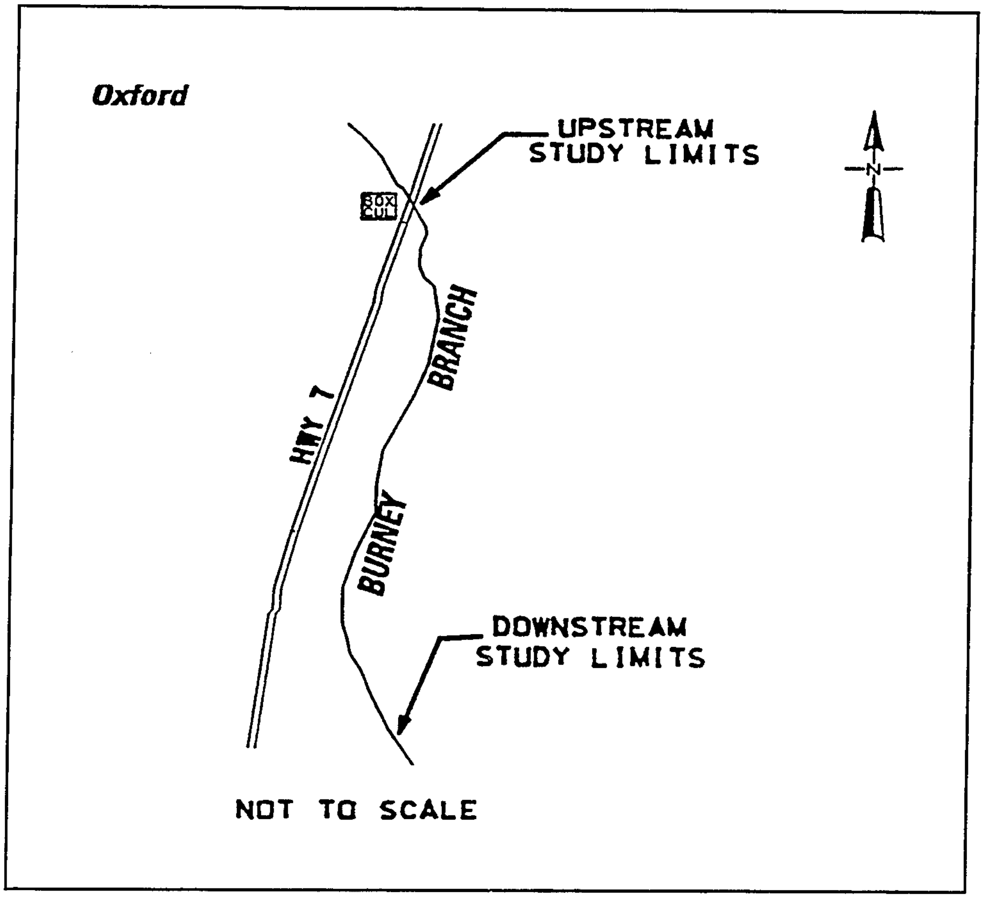

Figure 14. Burney Branch, Site 12

width is 113 percent of the slope and width required at minimum stream power for transport of 1,000 mg/ℓ. These dimensions generally indicate that the reach slope is near stability and bank erosion is due primarily to local hydraulic forces. Channel aggradation is occurring in the central reaches for approximately 4,000 ft. One reason for the aggradation may be an oversupply of coarser sediment to the stabilized sinuous reaches in which the tight bends may be causing hydraulic energy loss. Historic planform patterns in this reach indicated frequent cutoffs that are now prevented by willow post and bendway weir constructed features.

Observations made in the October 1994 field inspection indicated that the scalloping between bendway weirs has begun to be healed by colonizing vegetation, and filling between the riverward tips of the bendway weirs was observed. Willow post mortality has been high, and an overall survival rate of 42 percent was determined in the fall of 1994, down from an 80 percent survival in the spring of 1994. Survival rate improved for posts used in conjunction with 1 ton-per-linear-foot riprap toe; however, mortality rate was very high landward of the riprap toe if the landward fill did not drain adequately. See the section on bioengineering applications for DEC in Chapter 6.

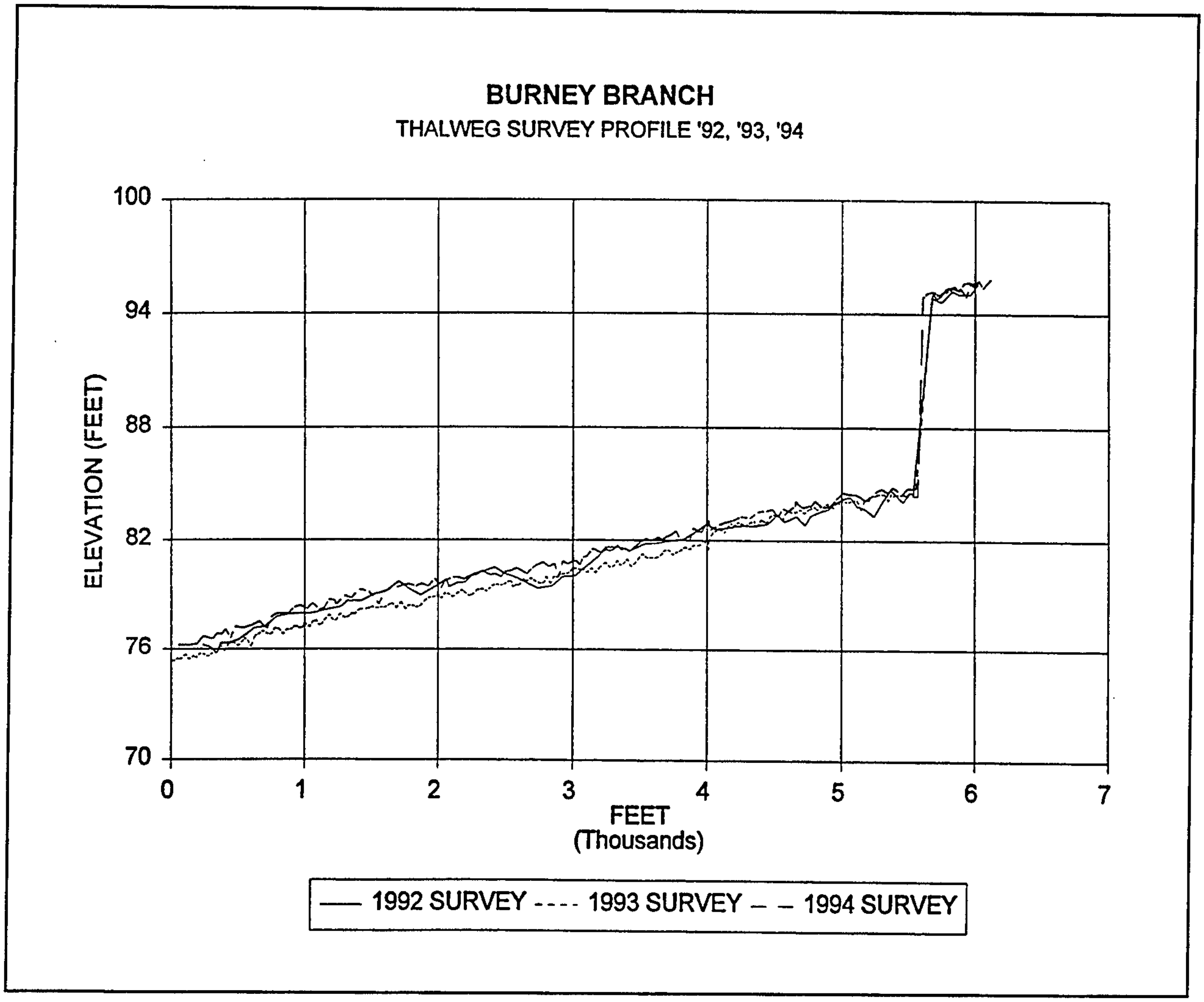

Figure 15. Burney Branch, Site 12, thalweg profile

Hickahala Creek at the confluence of South Fork Hickahala, Site 11

Hickahala Creek is a major tributary to the Coldwater River with a drainage area of approximately 230 square miles at the confluence with the Coldwater River. Field reconnaissance of channel geometry from 1968 and 1985 surveys and construction related surveys have also been conducted on upper Hickahala Creek in previous years. Stream gauge records are available from the USGS for a location near the mouth of the watershed.

Site 11 is located in the upper watershed of Hickahala Creek, and has a watershed area of approximately 9 square miles. The site is located on the Tyro quadrangle map in T5S, R5W, Sections 2 and 3, a portion of which is shown in Figure 22. The site begins at a county road bridge and extends downstream to the confluence with the South Fork, and continues downstream

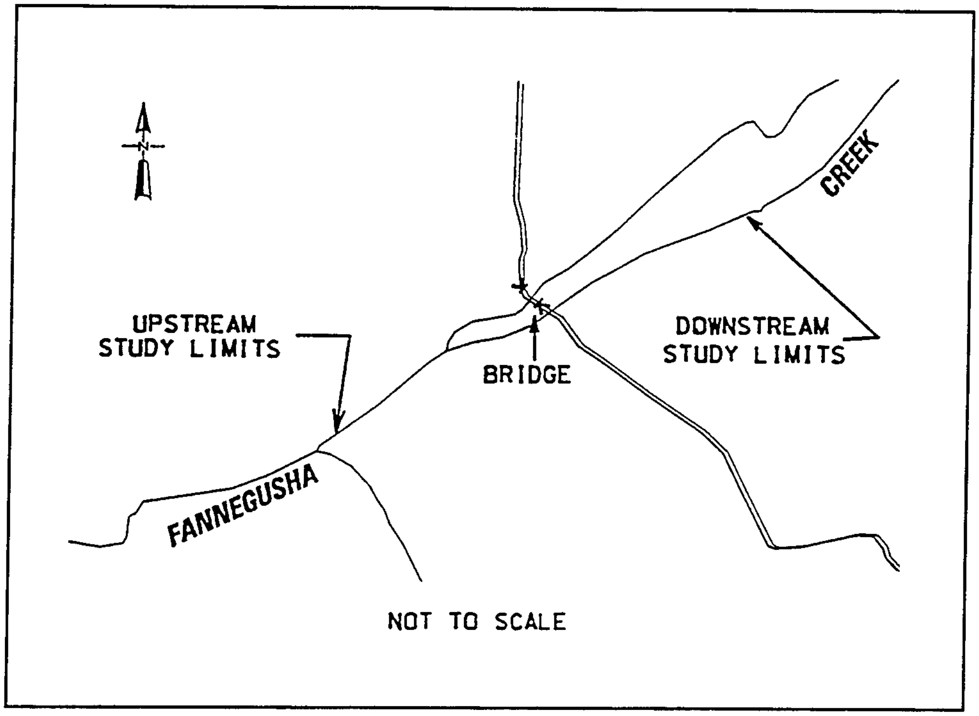

Figure 16. Fannegusha Creek, Site 2

on Hickahala Creek for approximately 1,000 ft. The total study reach is approximately 4,000 ft in length and includes two existing structures. A third structure is located on the South Fork about 700 ft upstream of the confluence with Hickahala Creek. The lower portion of the study reach is actively incising into a cohesive clay bed. The upstream portion of the study reach is relatively stable with a sand bed. The reach was selected to monitor the response of the structures.

Two low-drop structures are included in the study reach, and as shown in the accompanying thalweg profile (Figure 23), the downstream profile has been actively degrading. The upstream drop structure appears to have changed little during the last year, although some rock displacement upstream has occurred. The South Fork drop structure is a newer, grouted rock structure. Minor cracking of the grout was observed. A beaver dam, approximately 2 ft in height, has been built in the upstream approach riprap. The confluence of South Fork with Hickahala Creek is eroding badly, with high flows from Hickahala Creek now entering the tributary at a location about 150 ft upstream of the previous confluence. The confluence also has significant debris from fallen trees. The downstream structure on Hickahala Creek also has a beaver dam upstream of the structure. Water is ponded upstream, and local bank failures have occurred. This downstream structure is also a newer, grouted riprap structure. A significant crack, about 2 cm in width, exists along the left bank portion of the weir cap, between the grouted riprap and the concrete weir cap. Apparently, the grouted stone has rotated downstream to form this crack.

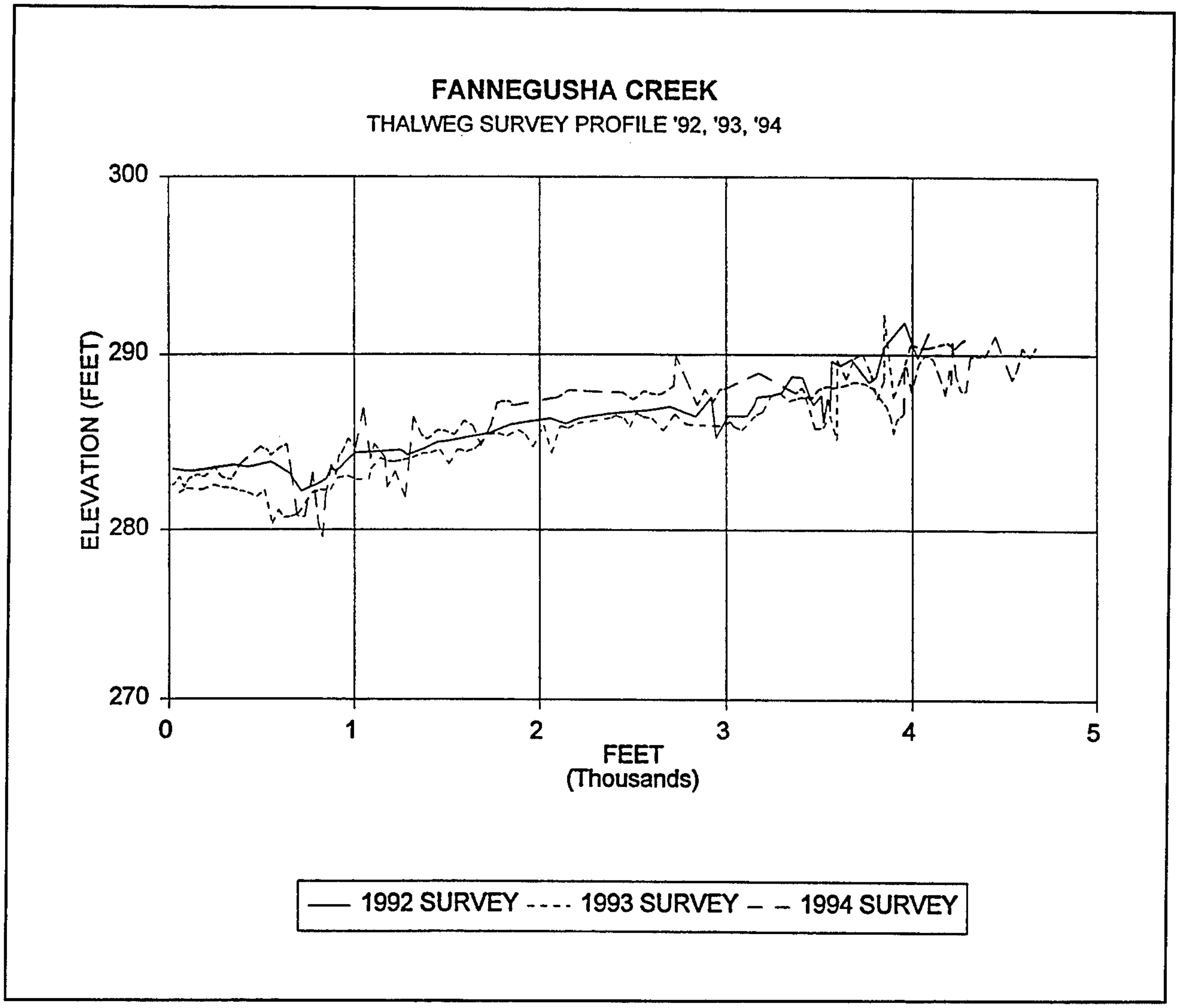

Figure 17. Fannegusha Creek, Site 2, thalweg profile

No similar feature was noted on the right bank. The channel is irregular downstream and a large headcut is located at approximate station 15+00.

The structures have given this site an opportunity to stabilize. Upstream of the South Fork structure and the upstream Hickahala Creek structure has significantly stabilized. A box culvert for the downstream bridge will help in stabilizing the downstream reach; however, the site downstream of the two upstream structures is generally unstable and adjustment will continue.

Field inspection of the site during November 1994 was conducted during the reconstruction of bridges at the upstream and downstream extent of the study reach. Drop pipes are presently located upstream of the upstream bridge and the bridge wing walls are very close to the pipes. The upstream bridge has been replaced with a double box culvert. The downstream bridge is located in

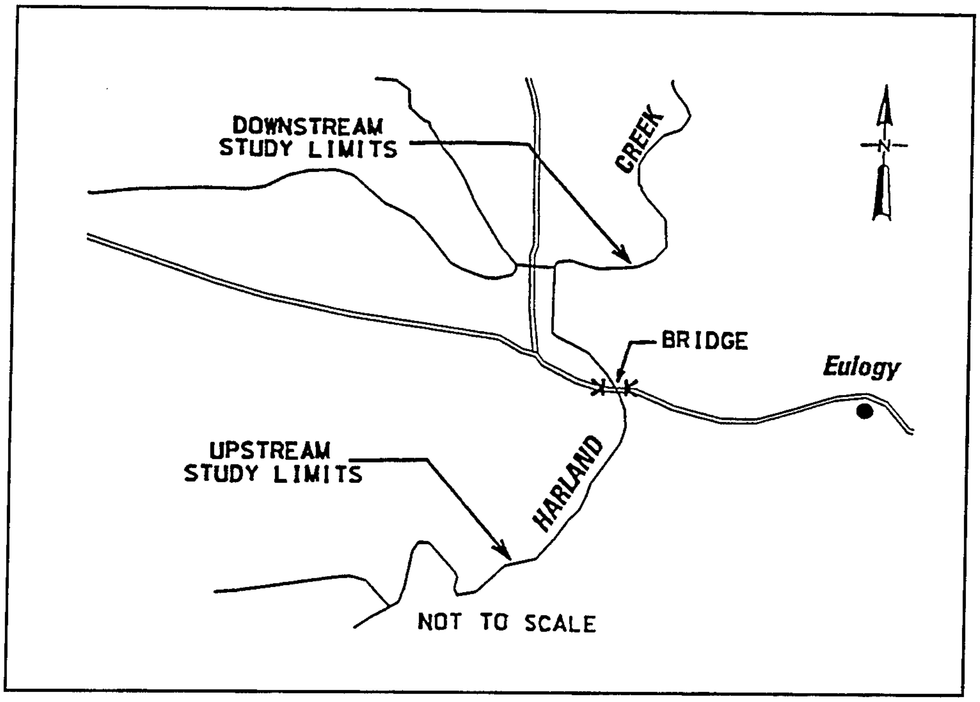

Figure 18. Harland Creek Site 1

a knick zone reach with a clay bed. The channel appears to be degrading, and it is hoped that the new downstream bridge will incorporate grade control.

Bank stability calculations indicate that the surveyed cross sections are generally stable and not at risk (Table 13). Based on the SAM analyses, segments 1 and 2 have continued to adjust. The 2-year water surface slopes in segments 1 and 2 were computed to be 134 and 137 percent, respectively, and the average stream widths are 182 and 167 percent, respectively, of the slope and width required at minimum stream power for transport of 1,000 mg/ℓ.

Hotophia Creek and Marcum Creek, Site 13

Site 13 is located on Hotophia Creek, west of Oxford. As shown in Figure 24, the site encompasses approximately 2 miles of Hotophia and Marcum Creeks and is located on the Sardis quadrangle map in T9S, R6W, Sections 1 and 2, and in T9S, R5W, Section 6. The watershed area at the site on Hotophia Creek is approximately 17 square miles. A USGS gauging station is located at the Highway 315 bridge crossing of the creek. The study reach includes the confluences of Marcum Creek and Deer Creek with Hotophia Creek. A low-drop is located at the downstream extent of Hotophia Creek and a high-drop is located on Hotophia Creek immediately downstream of the confluence with Marcum Creek. Two low-drops are situated on Deer Creek, and one low-drop is located on Marcum Creek approximately 800 ft upstream of the confluence with Hotophia Creek. Two additional high-drops, one within

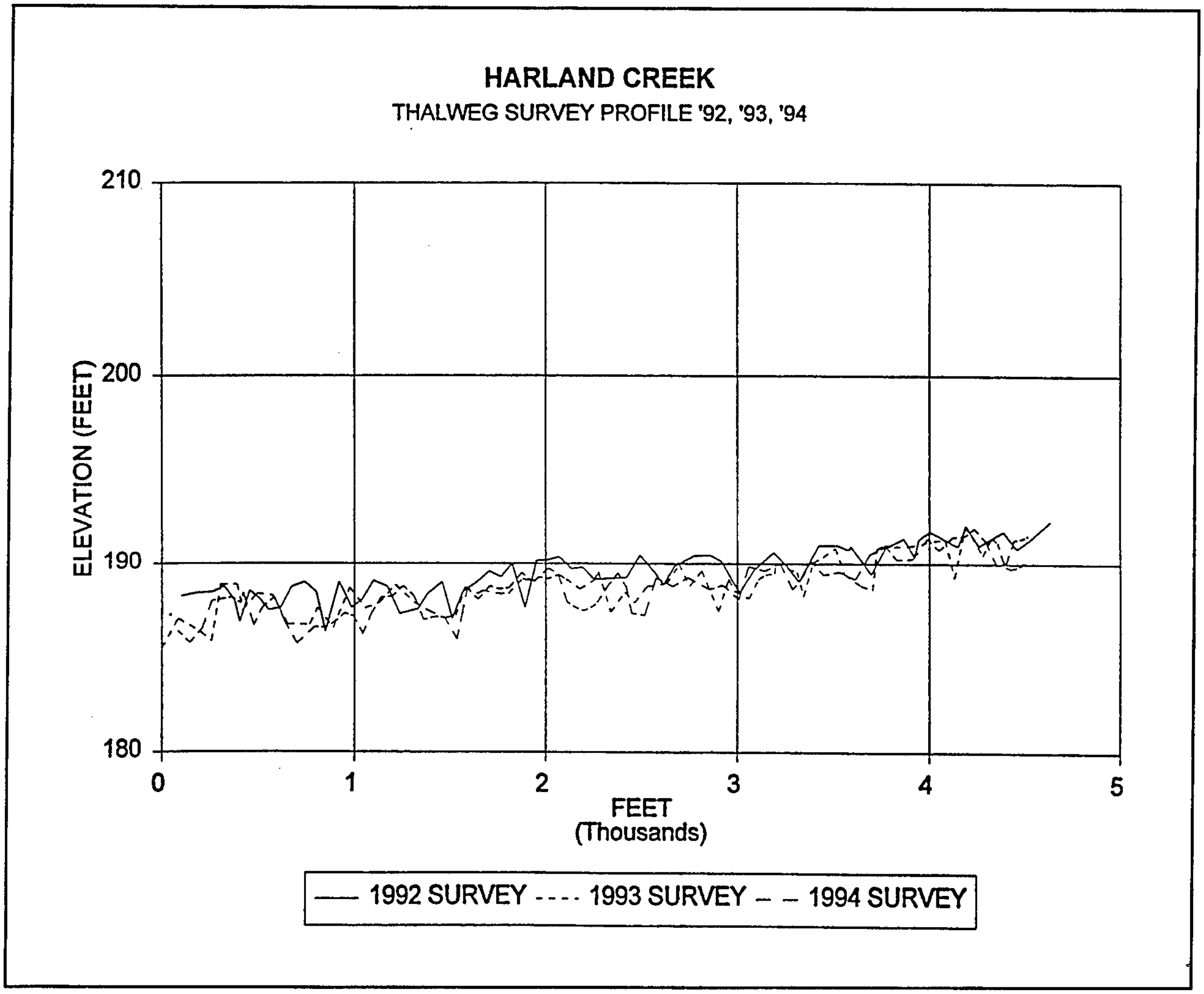

Figure 19. Harland Creek Site 1, thalweg profile

the reach and one upstream of the reach, were completed during 1994. WES-installed stream gauging is available at the first high-drop near the confluence of Marcum and Hotophia Creeks.

Hotophia Creek was channelized in 1961. This site is important because of the complexity of the various constructed elements and the need to document channel response to the high-drop grade control. In addition, data from Burney Branch and Hotophia Creeks provide the opportunity for a comparison of adjacent watersheds.

The primary change in the thalweg profile of Hotophia Creek during 1992-1994 is due to the construction of the downstream high-drop structure and the filling of the next upstream structure (Figure 25). The downstream extent of the study reach is the older low-drop structure, referred to as No. 8, which is downstream of Highway 315. This structure appears unchanged from last year, with a significant drop at the downstream extent of the riprap. However,

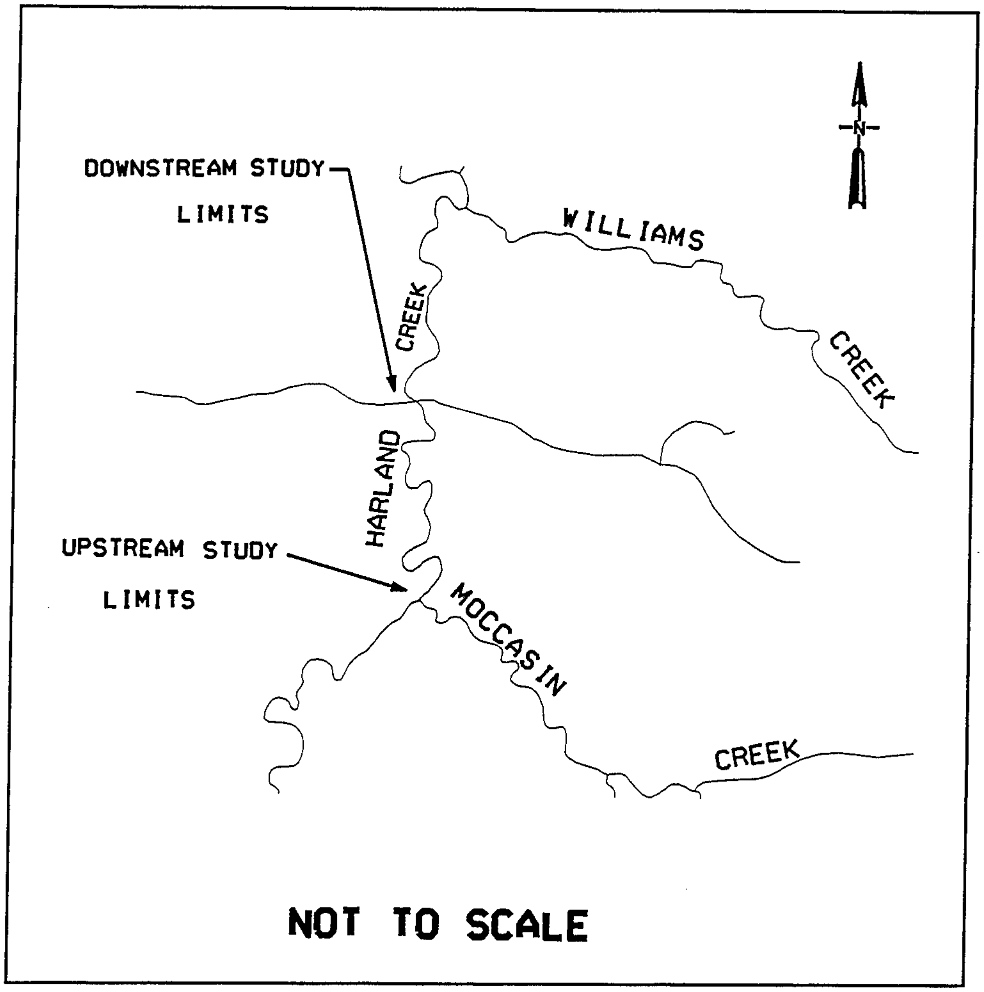

Figure 20. Harland Creek, Site 23

the water depth upstream seems to have increased from previous years and debris on the channel bed is visible from top bank looking down through the water column. A severe gully was observed on the left bank immediately downstream of the Highway 315 bridge and adjacent to an existing drop pipe. Apparently, the flow has bypassed the drop pipe and is causing a new gully along the downstream edge of existing riprap. Immediate attention is needed at this site. Another gully within the riprap is apparently causing sediment accumulation upstream of the high-drop structure. The channel flow is backwatered to the next high-drop, with little change in the channel. Sediment has accumulated upstream of the second high-drop down to the Marcum Creek confluence, with a slightly meandering channel forming in the newly aggraded sediment. Marcum Creek is inundated to within 300 ft of the Marcum Creek low-drop.

Marcum Creek has continued to degrade upstream of the Hotophia Creek high-drop influence, with the slope reducing from 378 to 320 percent of the slope at minimum stream power for the transport of 1,000 mg/ℓ (Table 14).

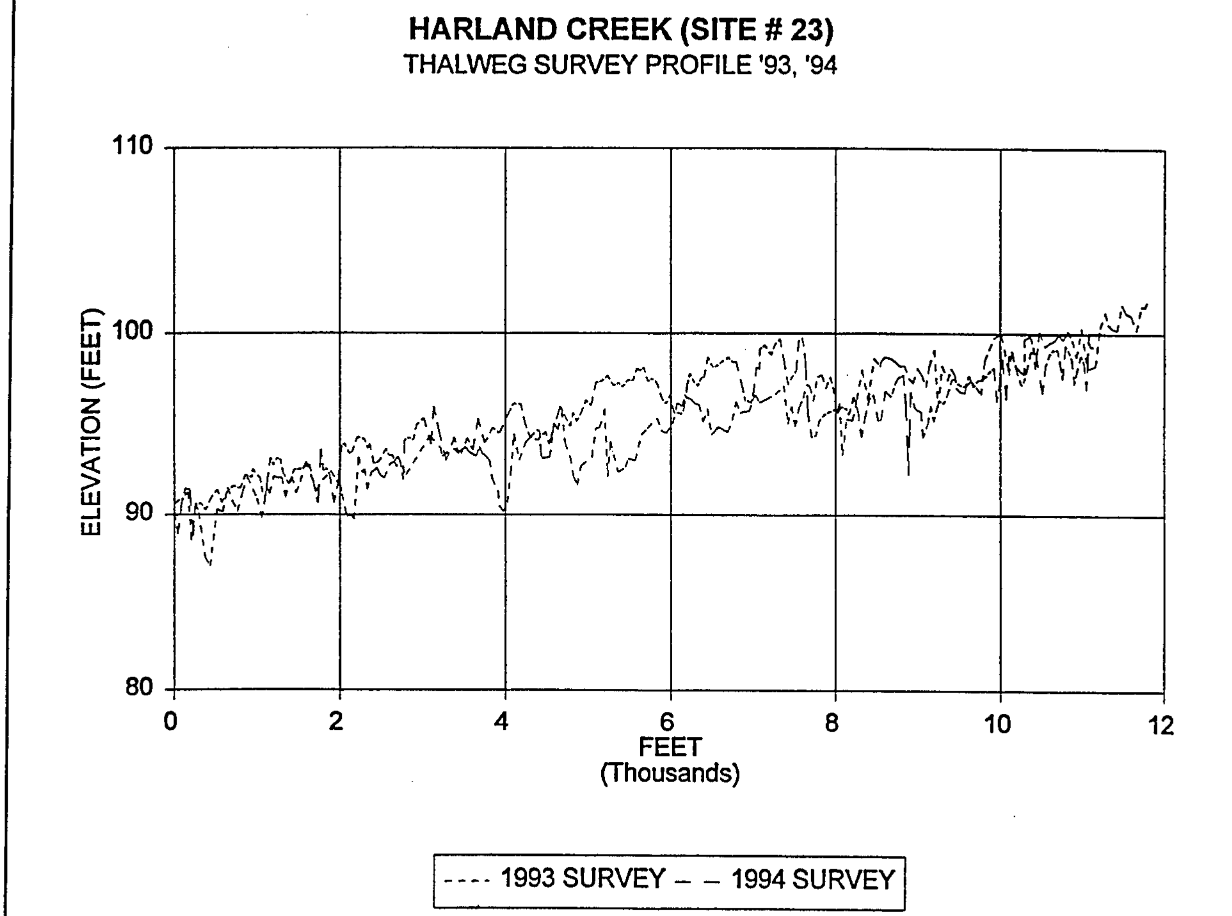

Figure 21. Harland Creek, Site 23, thalweg profile

Width is 88 percent of the width required for minimum stream power at the same sediment load. As shown in Table 14, width has increased and slope has decreased for Hotophia Creek, and the channel is approaching minimum stream power characteristics for the transport of 1,000 mg/ℓ. These channels are expected to continue to adjust toward equilibrium with the control imposed by the drop structures. Degradation downstream of structure No. 8 could continue.

James Wolf Creek, Site 19

Site 19 is located in the Hickahala Creek watershed on James Wolf Creek. At this location, James Wolf has a drainage area of approximately 11 square

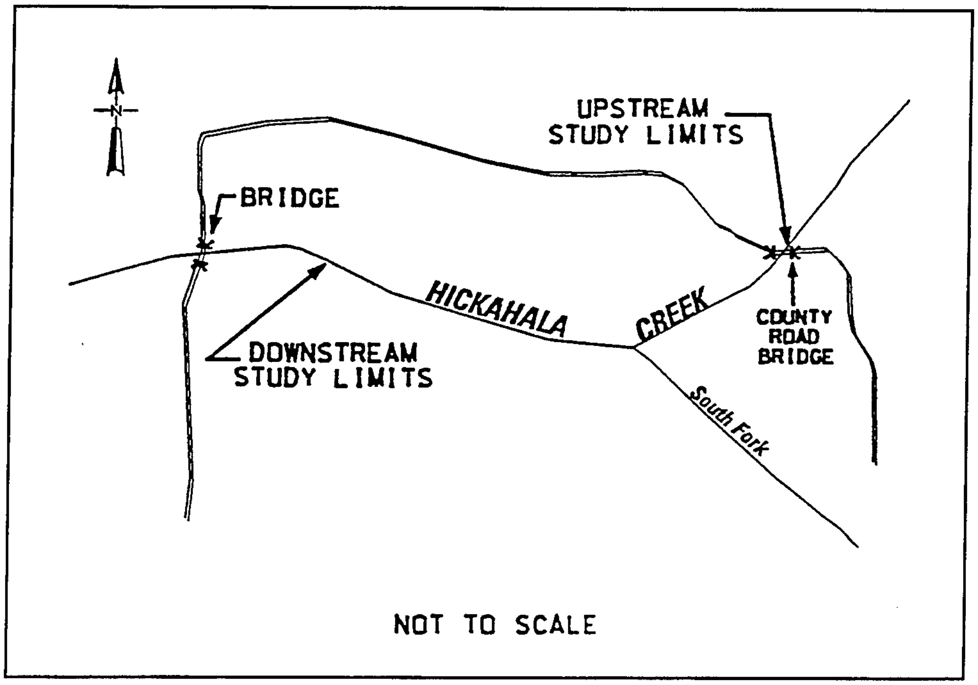

Figure 22. Hickahala Creek, Site 11

miles; however, it is extremely deep and wide. The site is located on the Tyro quadrangle map in T5S, R5W, Section 28, and extends downstream of the east-west county road for a distance of approximately 4,000 ft encompassing a low-drop structure (Figure 26). This low-drop structure appears to be stabilizing the bed of the stream; however, the banks remain unstable due to the significant depth. The stream has a sand bed, and at low-flow conditions, the channel may be dry. The drop structure has required significant repair since construction and is presently in need of significant repair. Two additional drop structures were constructed on James Wolf Creek downstream of the monitoring reach during 1993 and 1994.

The thalweg profile (Figure 27) indicates that no significant change has occurred during the past three surveys. The upstream scour feature has completely filled for the 1993 and 1994 surveys. A large beaver dam exists on the upstream riprap approach. Apparently, the beaver prefer the riprap foundation for their construction site. A backwater condition exists from the beaver dam to the upstream tributary on the left bank at approximate station 33+00. About 250 ft downstream from the upstream bridge, willows are establishing on an island in the center of the channel. Heavy kudzu growth dominates the bank vegetation. Riprap movement at the structure has been severe over the years and is continuing. No movement or cracking of the weir cap was noted.

BURBANK analysis of the bank stability indicates that 100 percent of the downstream and 54 percent of the upstream banks were unstable (Table 15). The improved bank stability is a result of installation of the grade control

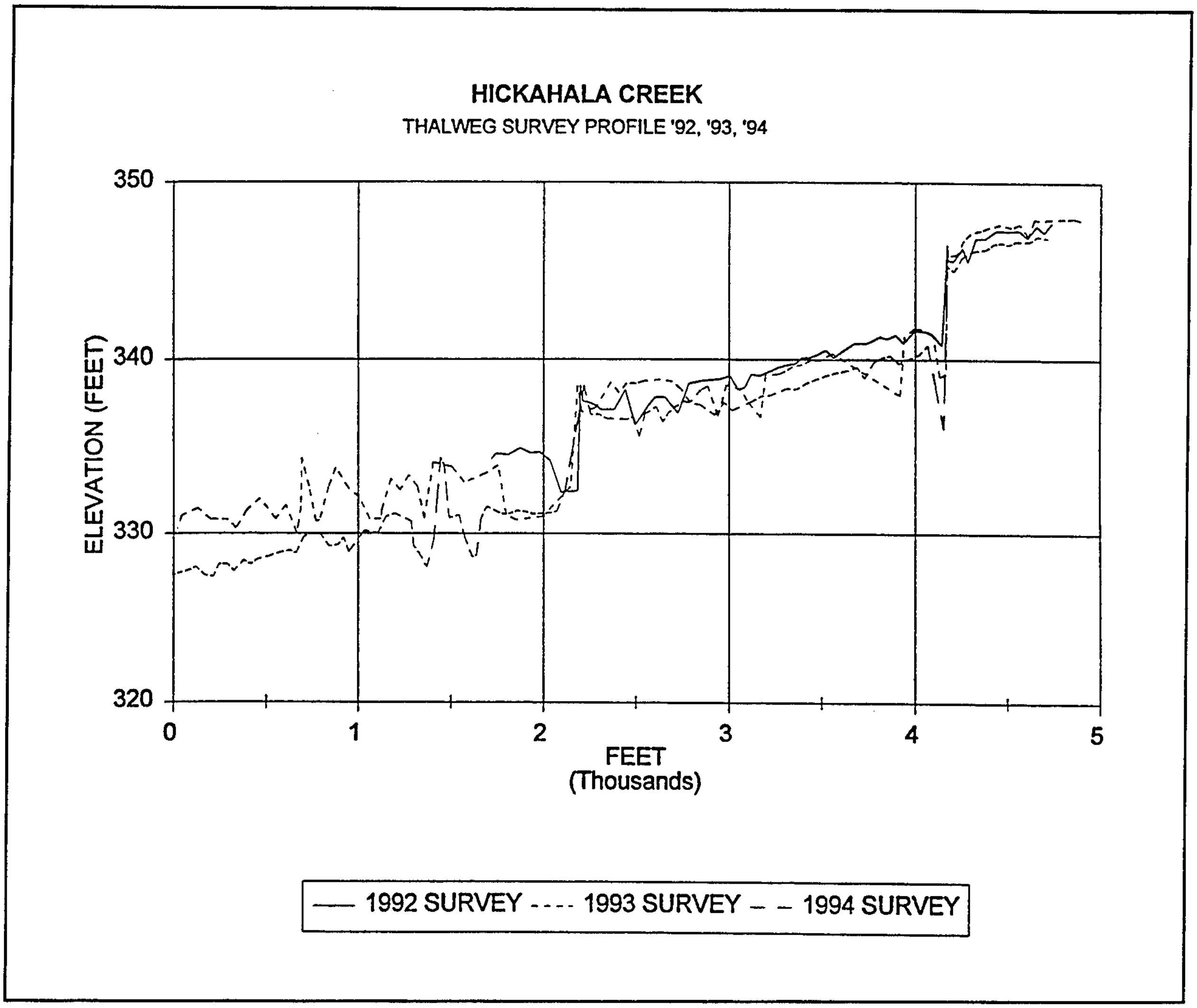

Figure 23. Hickahala Creek, Site 11, thalweg profile

structure in decreasing bank height. The 2-year water surface slope is 198 percent for segment 1 and 244 percent for segment 2, and the width is 150 percent for segment 1 and 98 percent for segment 2 of the slope and width required at minimum stream power to transport 1,000 mg/ℓ. The relatively high slopes indicate that the sediment supply from upstream is high. Reduction in sediment supply from upstream would reduce the channel slopes, but could result in renewed degradation, by flattening channel slope upstream of the structure which would cause increased bank instability upstream of the structure. To maintain the existing bank line, the bed should be raised as sediment supply decreases.

Lee Creek, Site 10

Site 10 is on Lee Creek in the Coldwater River basin, approximately 6 miles north of Victoria, MS. The site can be located on the Byhalia quadrangle map

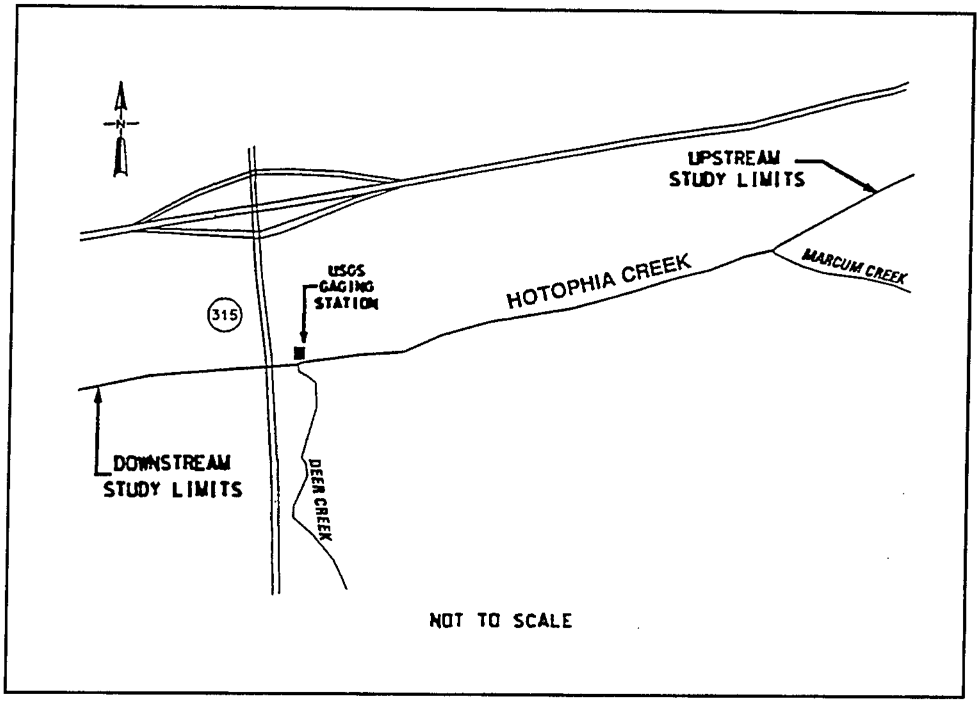

Figure 24. Hotophia Creek, Site 13

in T2S, R4W, Sections 9 and 10 (Figure 28). The study reach extends approximately 2,000 ft upstream and downstream of the highway bridge. The channel is relatively stable and is transporting minor amounts of gravel in a sand bed. Upstream of the bridge, the channel exhibits some meandering and apparently has not been channelized. Downstream of the bridge, the channel is stable with mature, 14-in.-diameter trees near the low-water surface. The remnants of spoil piles indicate that downstream of the bridge, the channel has been channelized. This reach provides an excellent opportunity to document a stable, channelized, sand-bed stream.

During October 1994, the property owner at the Lee Creek site requested that a drop pipe be considered for the left bank in the field upstream of the bridge that is immediately upstream of the Lee Creek site. He also suggested that the channel flood conveyance be improved within the upstream portion of the Lee Creek site. The upstream portion of the study site is in a cotton field with the channel banks covered by kudzu. The downstream portion of the study site is in pasture with grassed banks and a birch tree canopy. The downstream banks are relatively stable and conveyance is good, which is in direct contrast to the upstream portion of the study site. Kudzu has killed most competing vegetation in the upper portion. Only willow trees on islands within this small channel have been able to compete. The islands grow, creating divided flow, collecting debris, and reducing conveyance. Channelization to match the downstream channel section, eradication of kudzu, and planting of a mixture of grasses and birch trees could duplicate the downstream portion of the site and improve conveyance. Bridge abutments at the road crossing in the center of the study site are also in poor condition.

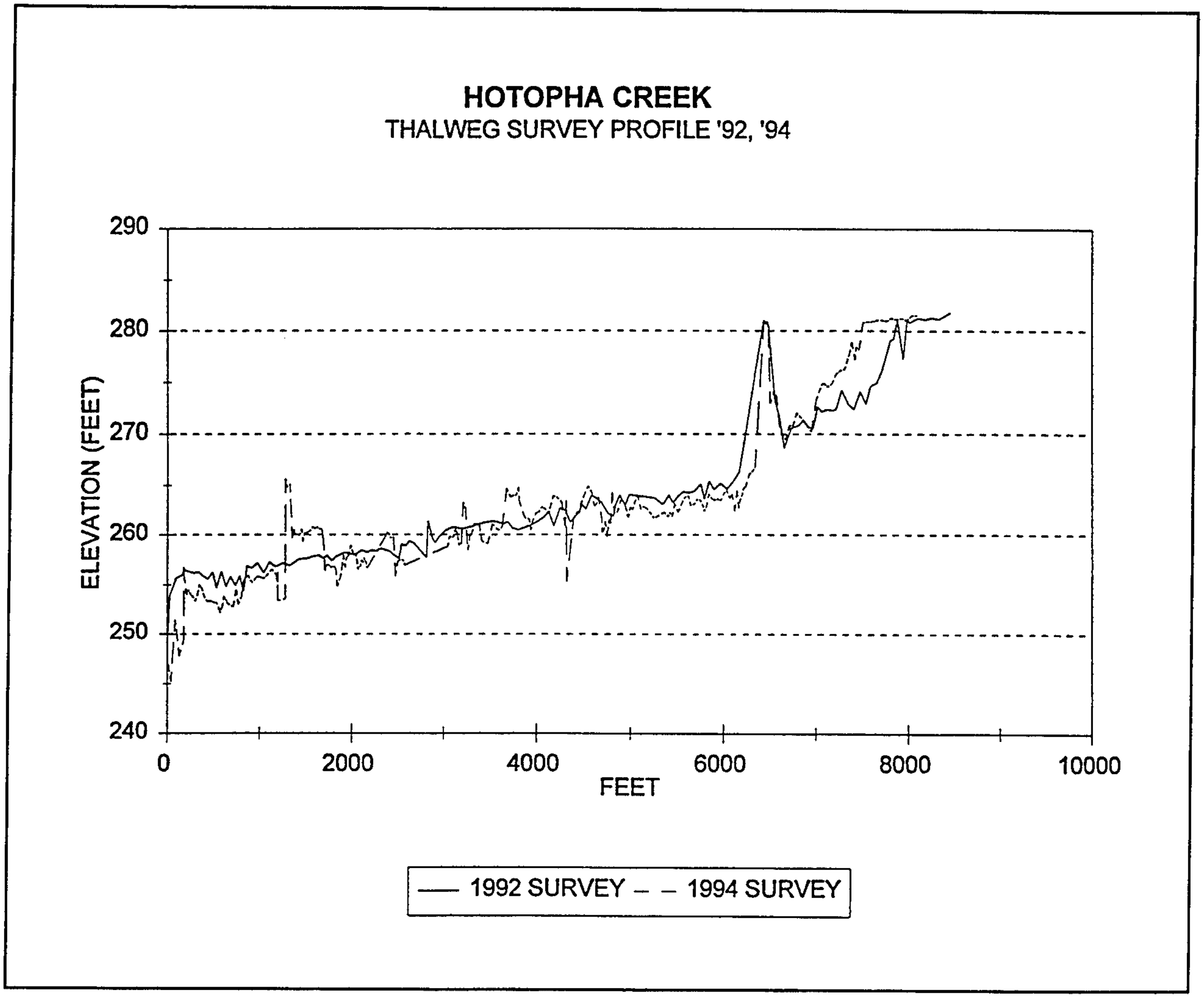

Figure 25. Hotophia Creek, Site 13, thalweg profile

Based on the channel profiles (Figure 29), from 1992 to 1994 channel degradation in the range of 2 to 4 ft occurred. Any channelization of the upstream portion of the site should include extending the survey upstream and consideration of grade control. BURBANK analysis confirms the field observations that the channel banks are stable. The 2-year water surface slope of the reach is 148 percent of the slope and 177 percent of the width for the width and slope required to transport 1,000 mg/ℓ at minimum slope, as defined by SAM (Table 16).

Lick Creek, Site 8

Site 8 is on Lick Creek in the Coldwater River basin, approximately 2 miles south of Olive Branch, MS. Construction of a high-drop structure was started in late 1994 to protect the Highway 305 Bridge. As shown in Figure 30, the study reach is approximately 4,000 ft in length, 2,000 ft

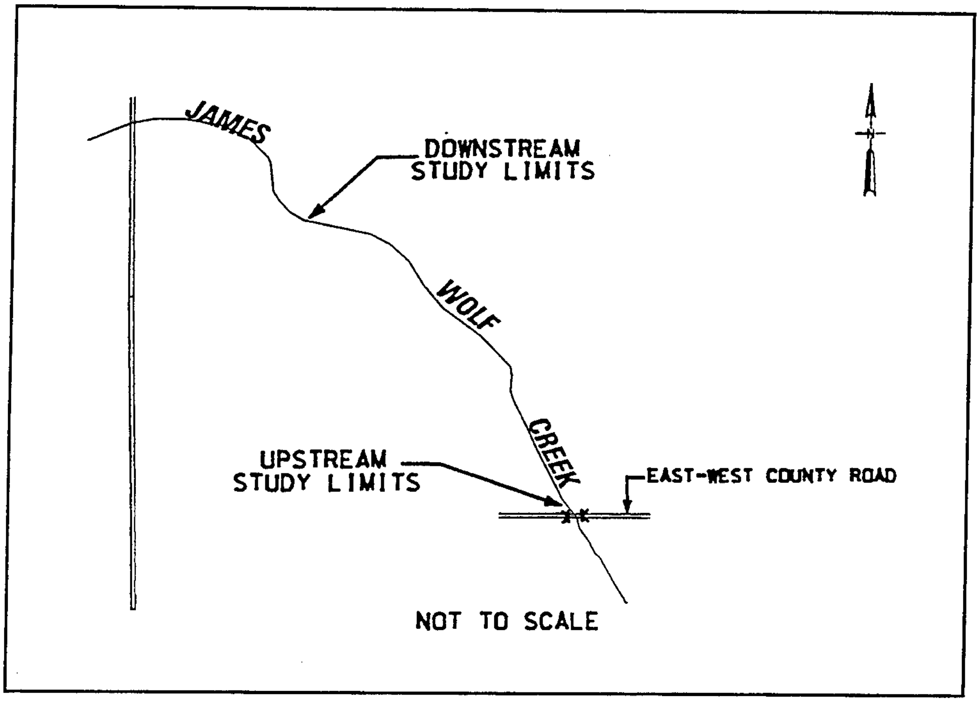

Figure 26. James Wolf Creek, Site 19

upstream and downstream of the bridge, in T2S, R6W, Section 3. This site is on the Hernando quadrangle map and has a watershed area of approximately 8.5 square miles.

The high-drop structure is located at approximate station 18+00 of the accompanying thalweg profile (Figure 31). Riprap placed at the bridge (station 20+00) as a temporary measure during construction of the structure has slowed the incision that is continuing upstream and downstream of the bridge. Degradation is continuing downstream of the structure and can be expected to continue after the closure of the structure. Backwater from the structure should assist in halting the upstream incision if the knick zones have not progressed too far upstream to be affected by the high-drop. The high-drop structure will protect the highway bridge. Presently, the upstream extent of the site is incising into resistant clay.

BURBANK analysis indicates that 3 ft of additional degradation will destabilize 19 percent of the surveyed banks. Left bank drainage upstream of the bridge is poor, with standing water in the adjacent field. Channel incision and a saturated left bank may combine to result in greater instability than in other similar streams. A drop pipe could be added to improve the bank drainage in that area. The SAM analysis indicates that the width has been increasing and the slope has been decreasing since the initial 1992 survey.

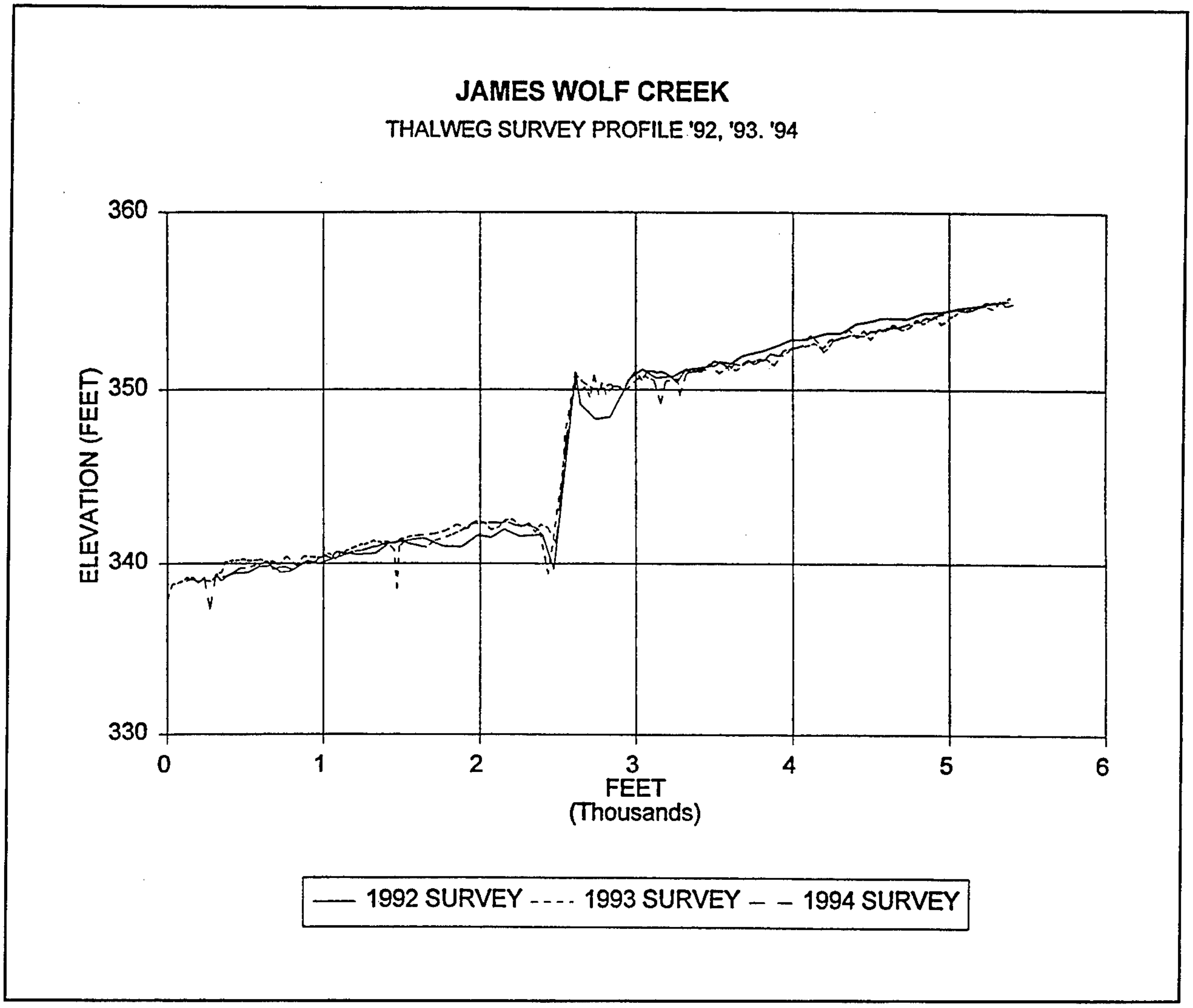

Figure 27. James Wolf Creek, Site 19, thalweg profile

Analysis of the 1994 survey indicates that the width is 188 percent and the slope 255 percent of the width and slope required at minimum stream power for transport of 1,000 mg/ℓ (Table 17). The high-drop structure will improve the stability of the upstream channel reach, and it will be of interest to observe the upstream and downstream channel response following completion of construction.

Long Creek, Site 20

Site 20 is located on Long Creek, T10S, R6W, Sections 4, 5, and 8, as shown in Figure 32. The site can be found on the Oakland quadrangle map and has a watershed area of about 11 square miles. Three low-drop structures existed prior to 1991 and the fourth was constructed in 1993 at the downstream limit of the monitoring reach. A fifth structure was constructed in 1993 downstream of the reach. The study reach is approximately 2 miles in length,

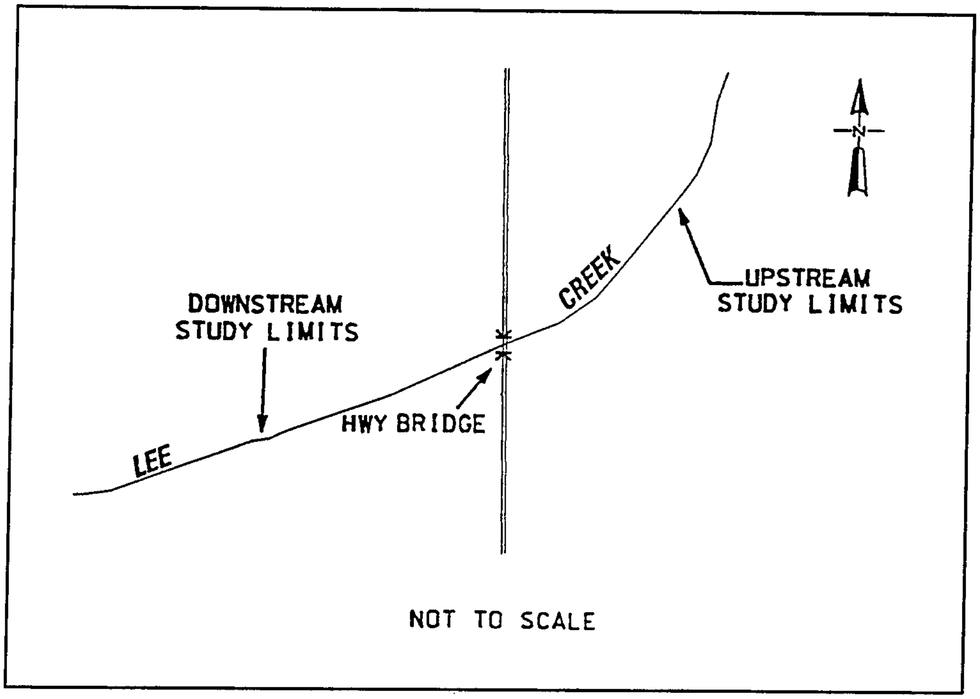

Figure 28. Lee Creek, Site 10

extending downstream from the eastern boundary of Section 4. The site also includes a reach that has been monitored by the Vicksburg District. Portions of the reach are very unstable and are presently incising. The reach downstream of the existing structures has a clay bed that was slowly incising prior to 1993. This clay bed was a very narrow, deeply incised channel along some reaches and has begun filling, a result of the new downstream structure.

Long Creek is divided into four segments: at station 0+00 at the fourth downstream drop structure; at approximate station 32+00 at the older low-drop structure; at approximate station 68+00 at the next upstream low-drop structure; and at approximate station 90+00 at the upstream structure. This structure at station 90+00 is a weir, not a low-drop structure, and is an at-grade sheet pile and concrete capped structure with no stilling basin. Figure 33 shows the thalweg profiles of Long Creek. Segment 1 aggradation has resulted since 1992 with the completion of the lower drop structure in 1993. The other significant thalweg change is within 300 ft downstream of the upper weir where headcutting is moving into the structure. The most significant change from 1993 is the number of beaver dams that are present in segments 2, 3, and 4.

BURBANK analysis shows the significant improvement in bank stability in segment 1 from 79 percent at risk in 1992 to 10 percent at risk in 1994 (Table 18). Bank instability in the remaining three segments is generally less than 10 percent. Without structural control, degradation would be continuing. The effects of 3 ft of degradation range from 69 percent in segment 1 to 25 percent in segment 3, which demonstrates one of the positive aspects of low-drop grade control. SAM analysis demonstrates the reduction in segment 1

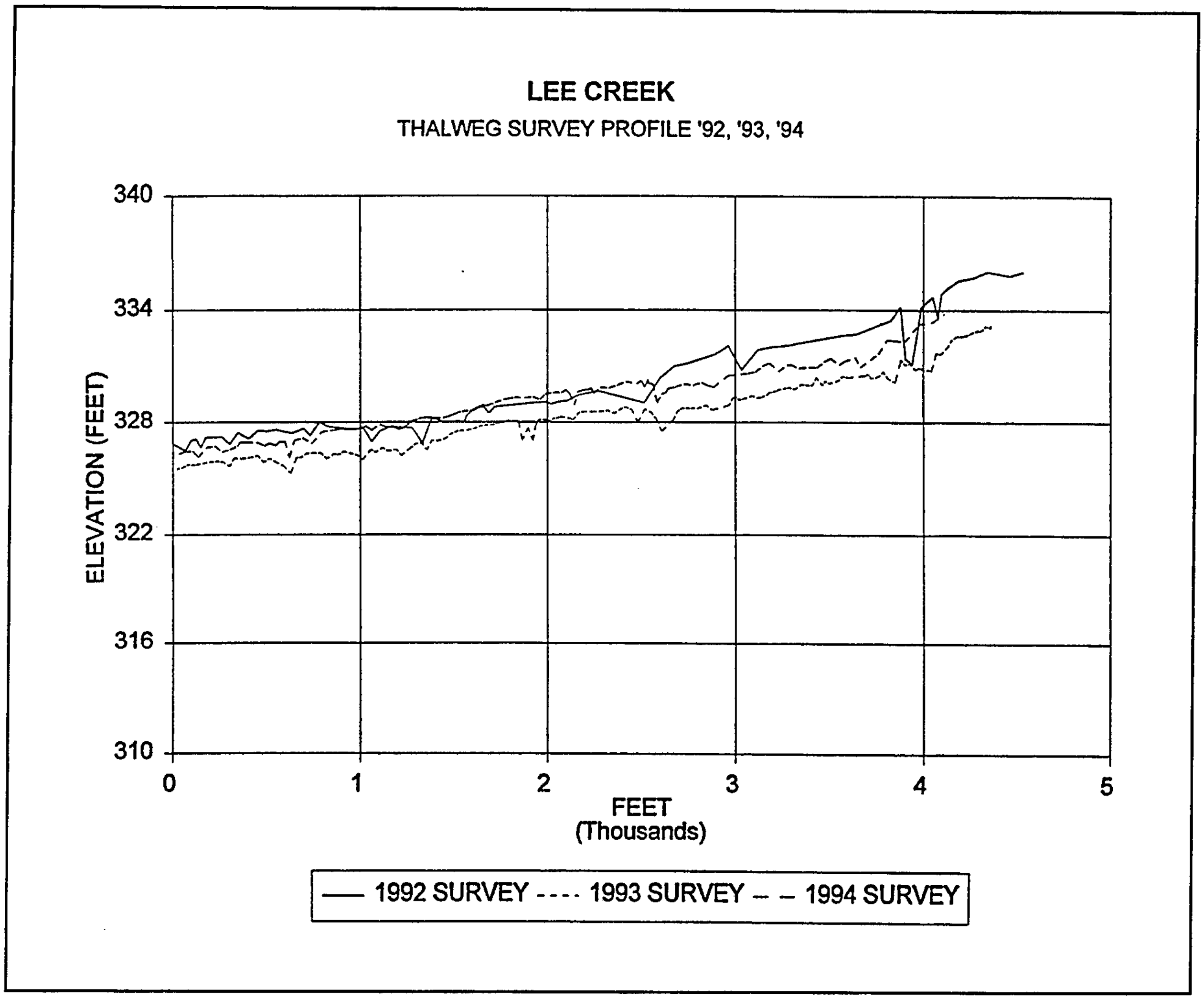

Figure 29. Lee Creek, Site 10, thalweg profile

slope, and shows the lower width in segments 3 and 4 where longitudinal riprap is placed on both channel banks almost continuously. The slopes for these two segments are approximately double the slopes for segments 1 and 2. Headcutting is present in segment 4. Monitoring of the long-term slope adjustment of the site will furnish unique information pertaining to channel adjustment in a channel that is limited in width adjustment. From an operational viewpoint, degradation is moving up to the upstream weir and should be monitored for the safety of the structure.

Nolehoe Creek, Site 7

Site 7 is located on Nolehoe Creek in the Coldwater River basin near the community of Olive Branch. The site is located on the Hernando quadrangle map, T1S, R7W, Section 35, and has a drainage area of approximately 3.7 square miles (Figure 34). The study reach is approximately 4,000 ft in

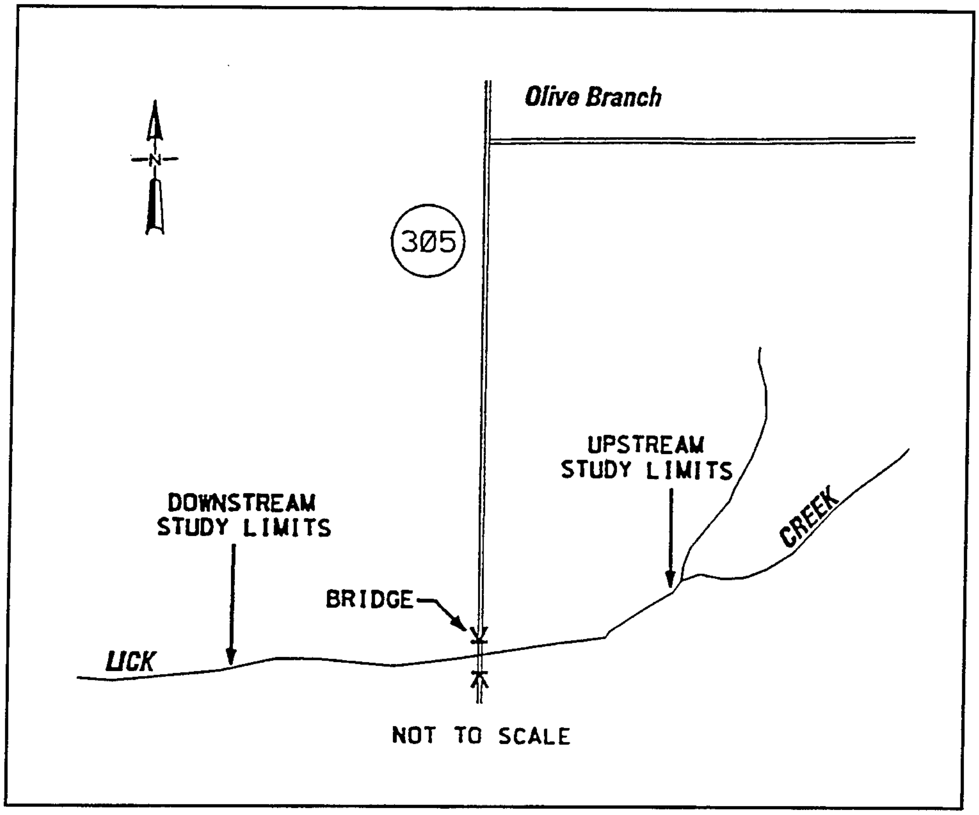

Figure 30. Lick Creek, Site 8

length, extending downstream from a box culvert. The channel is extremely unstable and is deeply incised. Bed material load ranges in size from fine sand to gravel with a mean diameter in excess of 30 mm. Two low-drop structures are planned for the reach; however, permission to construct the structures has not been received from the landowner. Stream stage recording stations have been recently installed by WES at the downstream roadway culvert.

This incising reach is between upstream and downstream box culverts, and the reach is representative of suburban development in the metro-Memphis area. An interview with a local landowner confirmed that a major cutoff of the channel had been made in the last 10 years. These conditions are typical of the result of ill-planned local development improvements, and the documentation of the resulting problems may be of value in assisting future local drainage planning.

As shown in the accompanying thalweg profile (Figure 35), Nolehoe Creek has continued to degrade by approximately 4 ft between 1992 and 1994. During the November 1994 site visit, construction personnel were replacing the upstream box culvert apron and wing walls. Apparently the recent degradation has caused the need to reinforce the apron, and erosion at and around the road side of each wing wall was severe. In addition to the construction at the site, the highway that follows an east-west route approximately parallel to Nolehoe

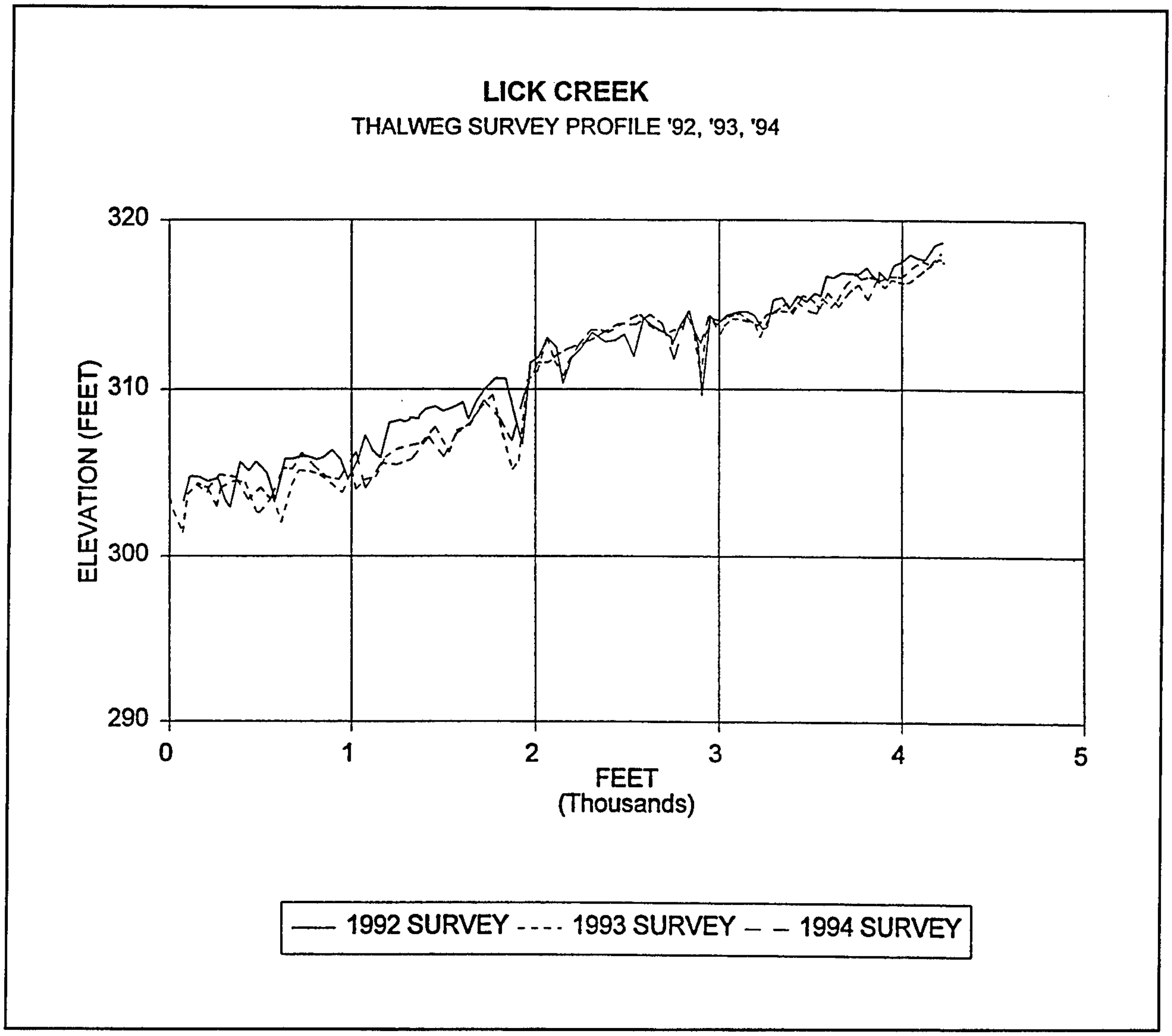

Figure 31. Lick Creek, Site 8, thalweg profile

Creek is in the process of being widened from a two-lane to a four-lane highway. Improvement adjacent to the highway includes better drainage, and the next upstream road crossing for Nolehoe Creek has been enlarged from a pipe culvert to a double concrete box. Residential development in the upstream watershed is continuing and the pattern of development is for commercial and office development along the four-lane highway. The downstream golf course and residential development portion of Nolehoe Creek has been staked, indicating that construction is planned. This portion of the creek is downstream of the study reach. Although no gauging or streamflow records are presently available, the changing land use suggests that discharges are increasing, and as the construction phase is completed, the urban land use pattern will supply less sediment in the future.

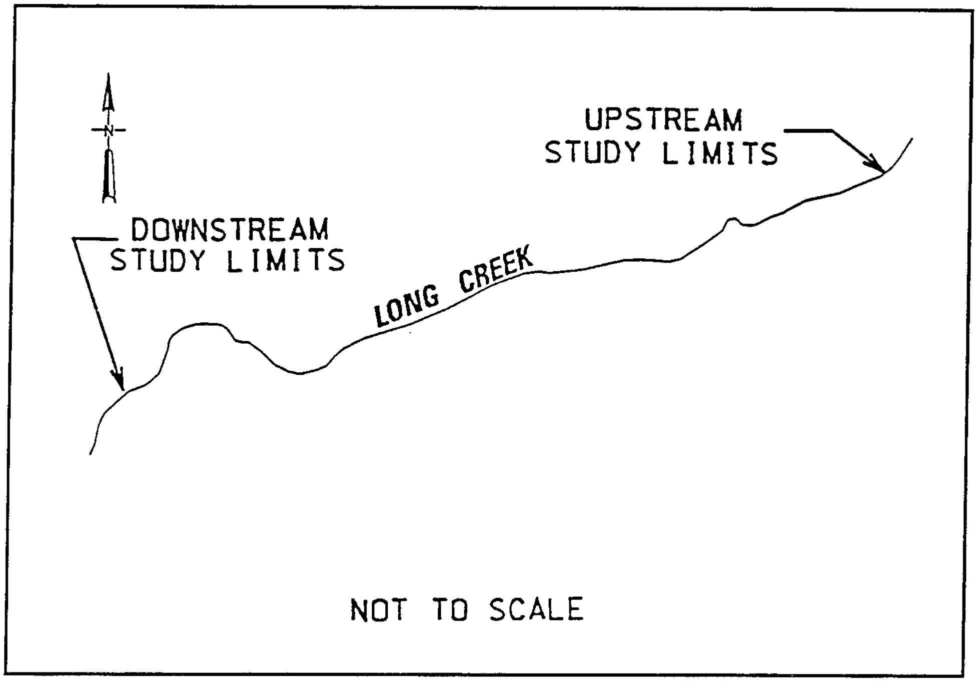

Figure 32. Long Creek, Site 20

BURBANK analysis indicates that the channel banks are becoming more unstable as incising continues, and that with 3 ft of degradation, approximately 30 percent of the channel banks would be unstable (Table 19). SAM analysis indicates that the channel width is approximately 113 percent of the width and the slope is 269 percent of the slope and width required at minimum stream power for transport of 1,000 mg/ℓ. Slopes have been decreasing for the last 3 years, from 295 to 269 percent. In the absence of grade control, the community should incorporate floodwater detention in drainage criteria.

Otoucalofa Creek, Site 14

Site 14 is on Otoucalofa Creek, east of Water Valley, MS. The study reach is 4,000 ft in length, 2,000 ft upstream and downstream of the Mt. Liberty Church road bridge, in T11S, R3W, Sections 4 and 5, of the Water Valley quadrangle map (Figure 36). The watershed area at the site is approximately 41 square miles.

Presently, only riprap dikes and longitudinal dikes are constructed throughout the reach; however, a low-drop structure is proposed for the future. The reach was observed to be actively incising, and this incision is occurring at an elevation below the recently placed stone. This site provides a unique opportunity to observe the riprap subjected to degradation.

As shown in the accompanying thalweg profile (Figure 37), Otoucalofa Creek is degrading and a steep knick zone exists just beyond the upstream

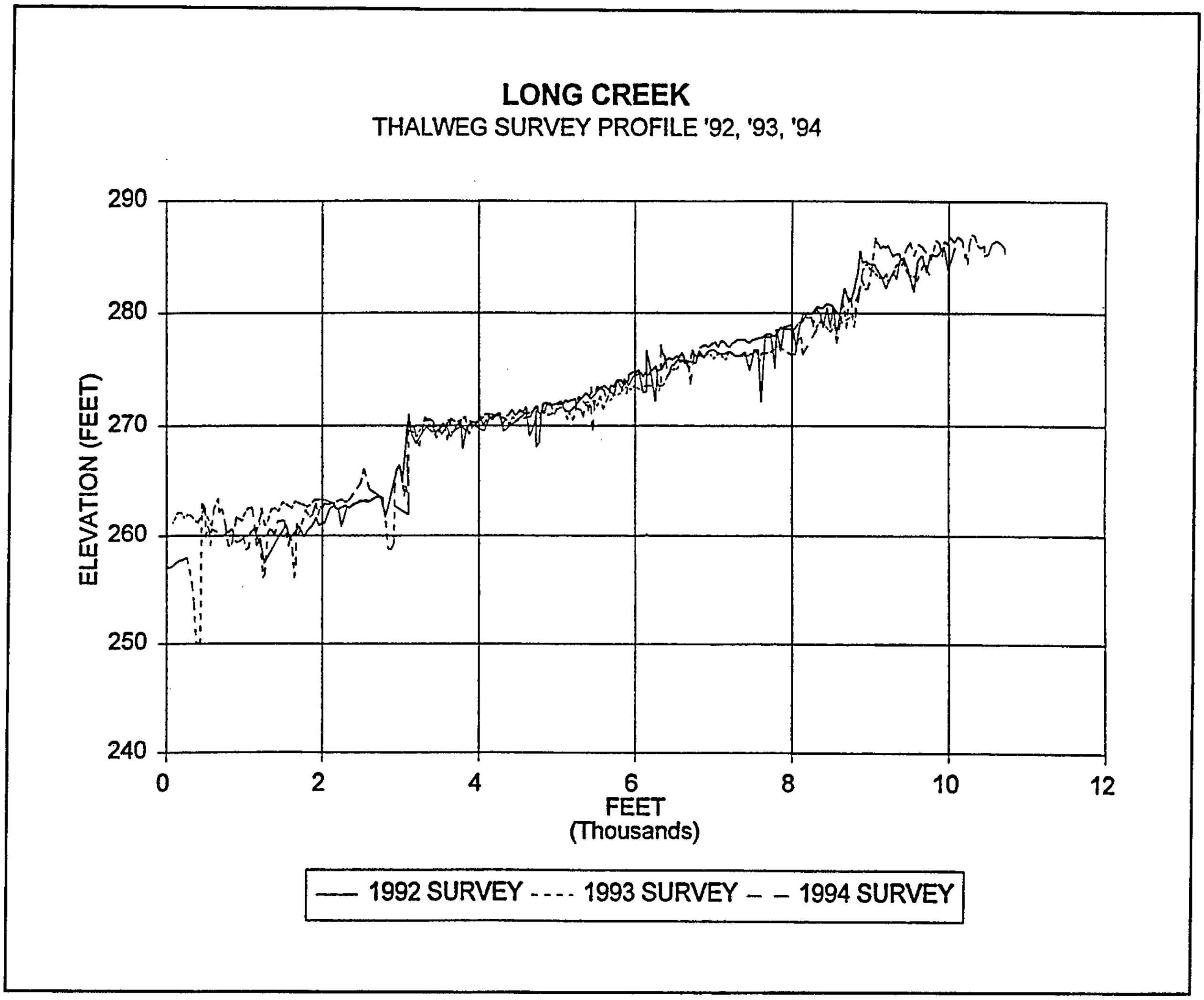

Figure 33. Long Creek, Site 20, thalweg profile

extent of the study reach. The Mt. Liberty Church road bridge is located at approximate station 20+00. Downstream of the bridge the channel is relatively wide and meandering with some point bar formation. The banks have been revetted, and only minor launching of the revetment has been observed. At the downstream extent of the reach, the left bank is protected by a series of dikes that are experiencing severe launching; however, the dikes remain functional. Upstream of the bridge, longitudinal toe riprap and dikes have been placed on what was the hard clay bed of the channel. Incision has progressed up through the prior bed and formed a narrow inner channel that is steep and active and generally below the riprap.

BURBANK analysis indicates that the banks are stable, which means that the previous failures at the time of the January 1994 survey resulted in a geotechnically stable bank (Table 20). Some of the locations of instability upstream of the bridge were not surveyed due to safety reasons; therefore, the data may not represent the most severe sites. SAM analysis indicates that the

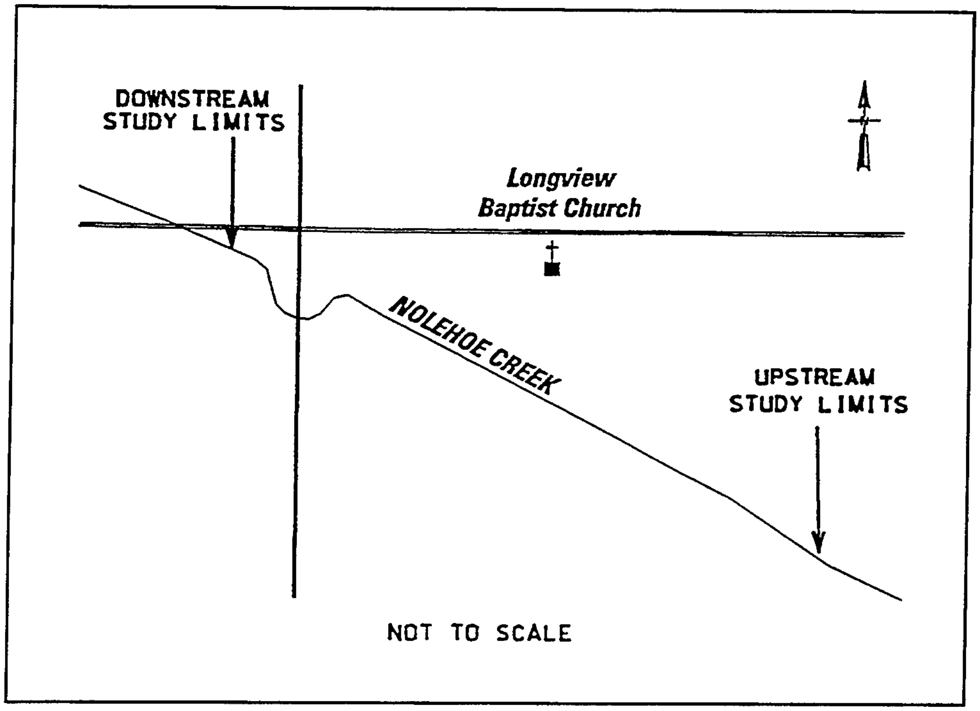

Figure 34. Nolehoe Creek, Site 7

average channel width is very narrow, 63 percent of the width required for minimum stream power with transport of 1,000 mg/ℓ. The slope is 141 percent of the similar value.

The vertical instability may be related to the narrowing of the portion of the reach upstream of the bridge by the longitudinal toe riprap and transverse dike construction. Along with Red Banks Creek, a longer reach of the stream may require analysis using HEC-6 to provide reasons for the instability.

Perry Creek, Site 16

Site 16 is located on Perry Creek as shown in Figure 38. The study reach begins approximately at the T21N, R4E, Section 1, northern line and continues upstream through Sections 2 and 11. The study reach is located on the McCarley quadrangle map. The entire study reach length is approximately 2 miles. Four low-drop structures were completed during 1994. This site will allow the investigation of the effects of four structures in series. The site is unique because within the study reach, the channel moves from a deeply incised stream at the downstream end to a stream that might have existed prior to channelization at the upstream end. Plans are to gauge the stream at the I-55 box culvert downstream of the study reach.

Field inspection in October 1994 indicates that vegetation density is increasing along the study reach and that the beavers have constructed numerous dams. Gullying was noted within the construction area and downstream of the

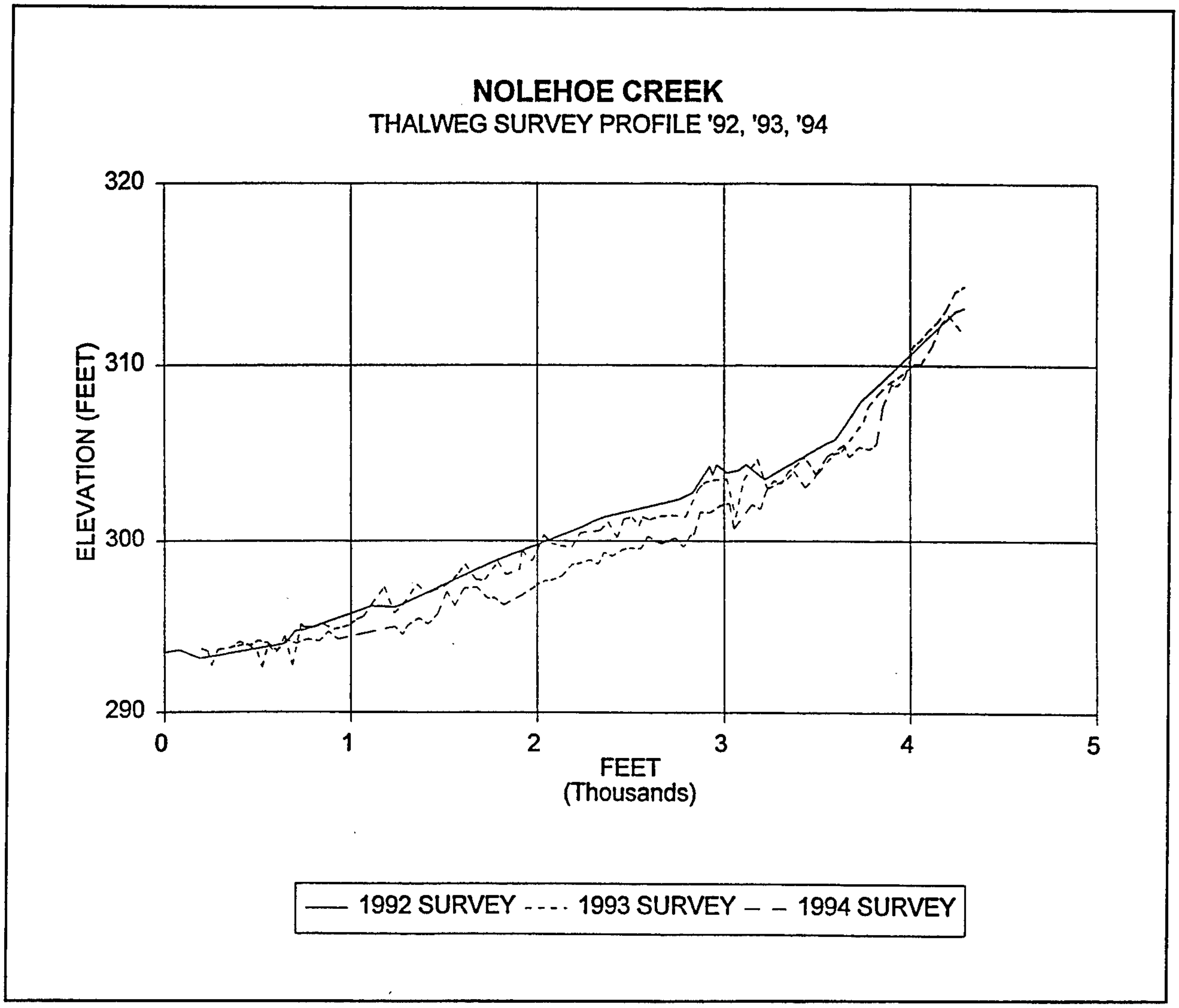

Figure 35. Nolehoe Creek, Site 7, thalweg profile

left bank riprap at the third structure. This gully enters the channel in the toe riprap. Another gully, which should be considered for a drop pipe, is present on the right bank downstream of the left bank existing pond and drop pipe, approximately adjacent to the power pole in the open field. The bed of the channel is very steep upstream of the third structure and composed of fractured ironstone and clay. Upstream of the fourth structure and within the construction clearing on the left bank is a severe gully that requires attention. In general, the structures are performing well and the system seems to be stabilized; however, specific sites need attention.

The accompanying thalweg profile (Figure 39) shows the response to the construction of the four grade control structures. Surveys during the 1993 and 1994 period represent construction conditions. The four structures are located at approximate station 20+00, 50+00, 85+00, and 100+00. However,

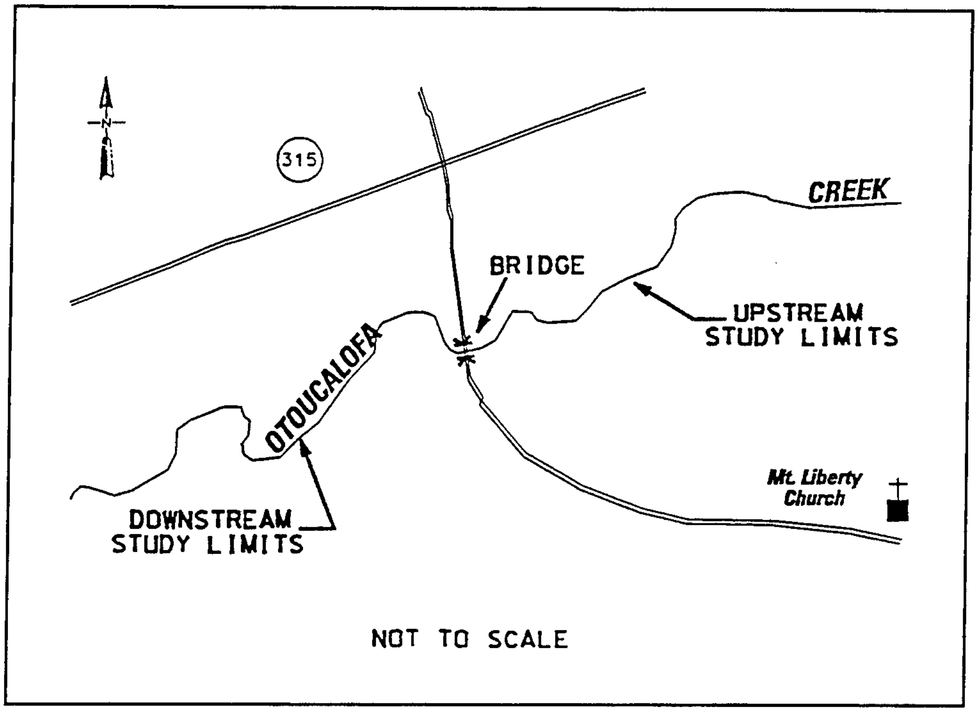

Figure 36. Otoucalofa Creek, Site 14

surveys prior to the ones conducted during 1995 did not provide cross sections significantly far enough upstream of the upstream structure to permit analysis.

Analysis of the bank stability, summarized in Table 21, indicates that the grade control structures have significantly reduced downstream sediment load by causing aggradation and reducing the probability of continued headcutting. For example, the BURBANK analysis indicates that 1 ft of degradation would result in approximately 160 cu yd and 3 ft of degradation would result in 265 cu yd of material contributed from bank failure in the study reach of Perry Creek. The 2-year water surface slope of the reach is decreasing, and the average stream width is increasing to approach the slope and width required at minimum stream power for transport of 1,000 mg/ℓ. These dimensions generally indicate that the reach morphology is approaching stability. Cross-section data were not available in 1992 for hydraulic analysis.

Red Banks, Site 9

Site 9 is located on Red Banks Creek in the Coldwater River basin (Figure 40). The study reach extends approximately 2.5 miles upstream from the bridge on the county road between the communities of Warsaw and Watson, MS. This site can be located on the Byhalia quadrangle map, T3S, R5W, Section 24, and R4W, Sections 19 and 20, and has a watershed area of approximately 28 square miles. The bed sediment load is sand, and the stream flows in a deeply incised and widened, straight channel that is the consequence

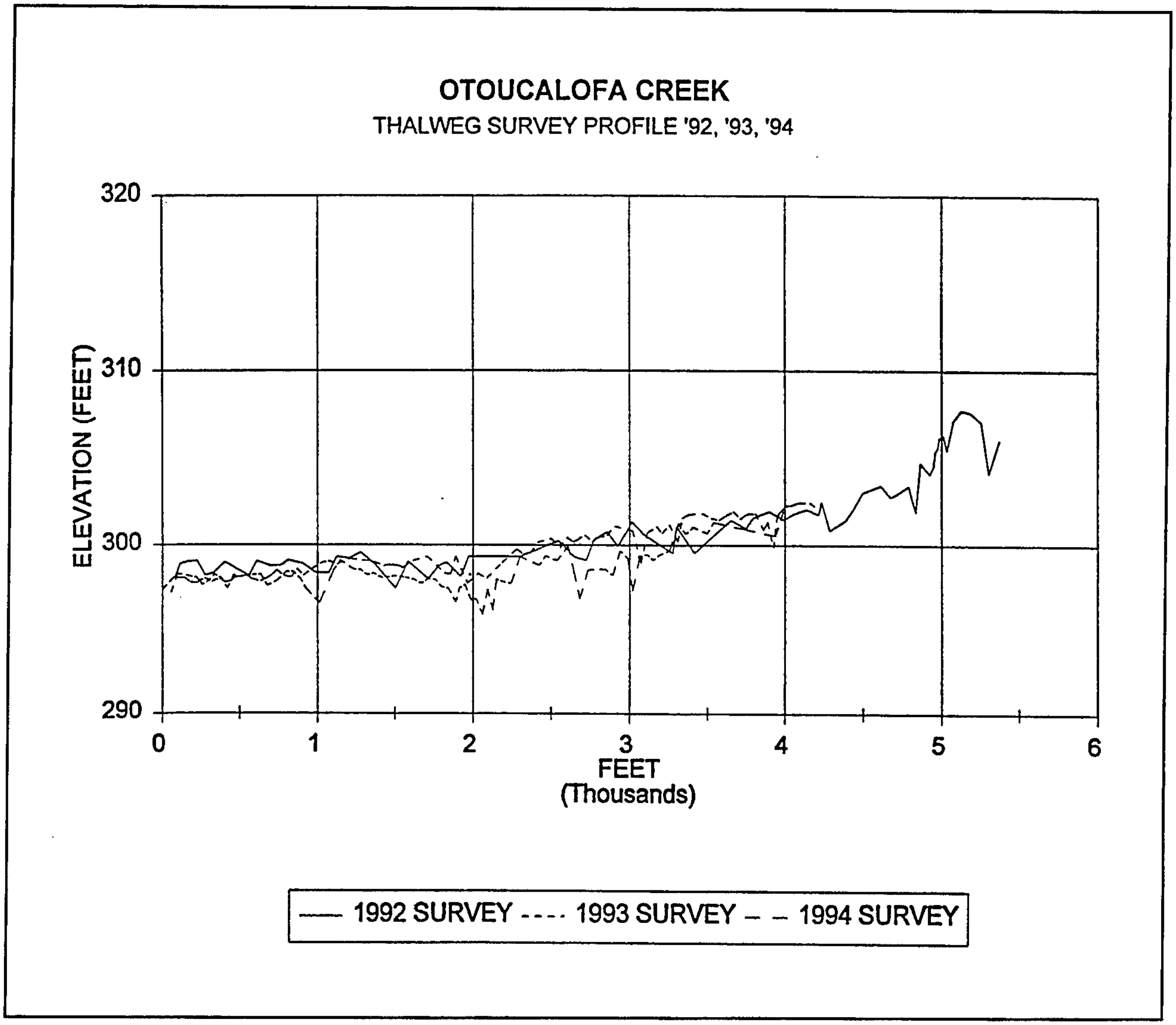

Figure 37. Otoucalofa Creek, Site 14, thalweg profile

of earlier channelization. Site 9 is unique in that it is the only DEC monitoring site using chevron dikes and longitudinal dikes for channel stabilization.

As shown in the accompanying thalweg profile (Figure 41), Red Banks Creek has been dynamic during the 1992 to 1994 study period. Prior to 1992, 2 tons per linear foot longitudinal riprap and four riprap chevron weirs were placed in the study reach. The channel was narrowed significantly at the downstream bridge and immediately upstream of the bridge for a distance of approximately 100 ft. Degradation in the range of from 1 to 3 ft is shown from 1992 to 1993 and to 1994 downstream of the chevron weirs. Approximately at station 50+00 a headcut was observed in 1992 and 1993. The pool downstream of the most downstream chevron weir was surveyed to be in excess of 10 ft in depth and increased in depth during the 1992 and 1993 period. During the 1994 thalweg survey, degradation had occurred and the riprap of this

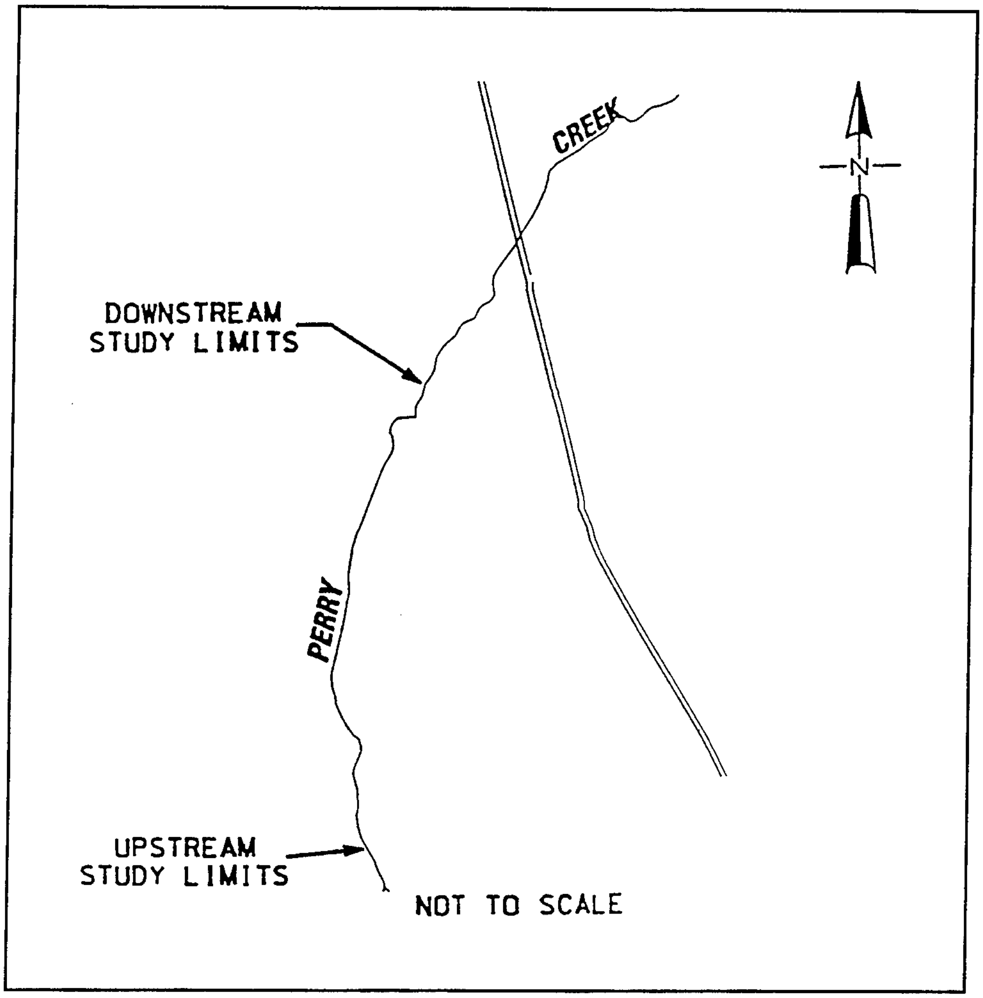

Figure 38. Perry Creek, Site 16

chevron weir was observed to be displaced. Less stone has been displaced in the second weir than in the first, and the two upper weirs exhibit significant displacement. The longitudinal riprap has remained stable with launching only in the reach downstream of the weirs.

BURBANK analysis indicates that in 1994 only 6 percent of the banks were at risk of failure; however, this would increase to 25 percent with an average degradation of 3 ft (Table 22). The 2-year water surface slope is 179 percent and the width is 85 percent of the slope and width required at minimum stream power to transport 1,000 mg/ℓ. The width has remained relatively constant for the three surveys; however, the slope increased from 180 percent in 1992, to 199 percent in 1993, then back to 179 percent in 1994. Increase in slope from 1992 to 1993 could have resulted from increased differential in bed elevation upstream and downstream of the weirs and from adjustments in channel morphology. Undoubtedly, the combination of longitudinal riprap and upstream aggradation resulted in a wider channel upstream than downstream of the weirs as the channel aggraded upstream and degraded downstream.

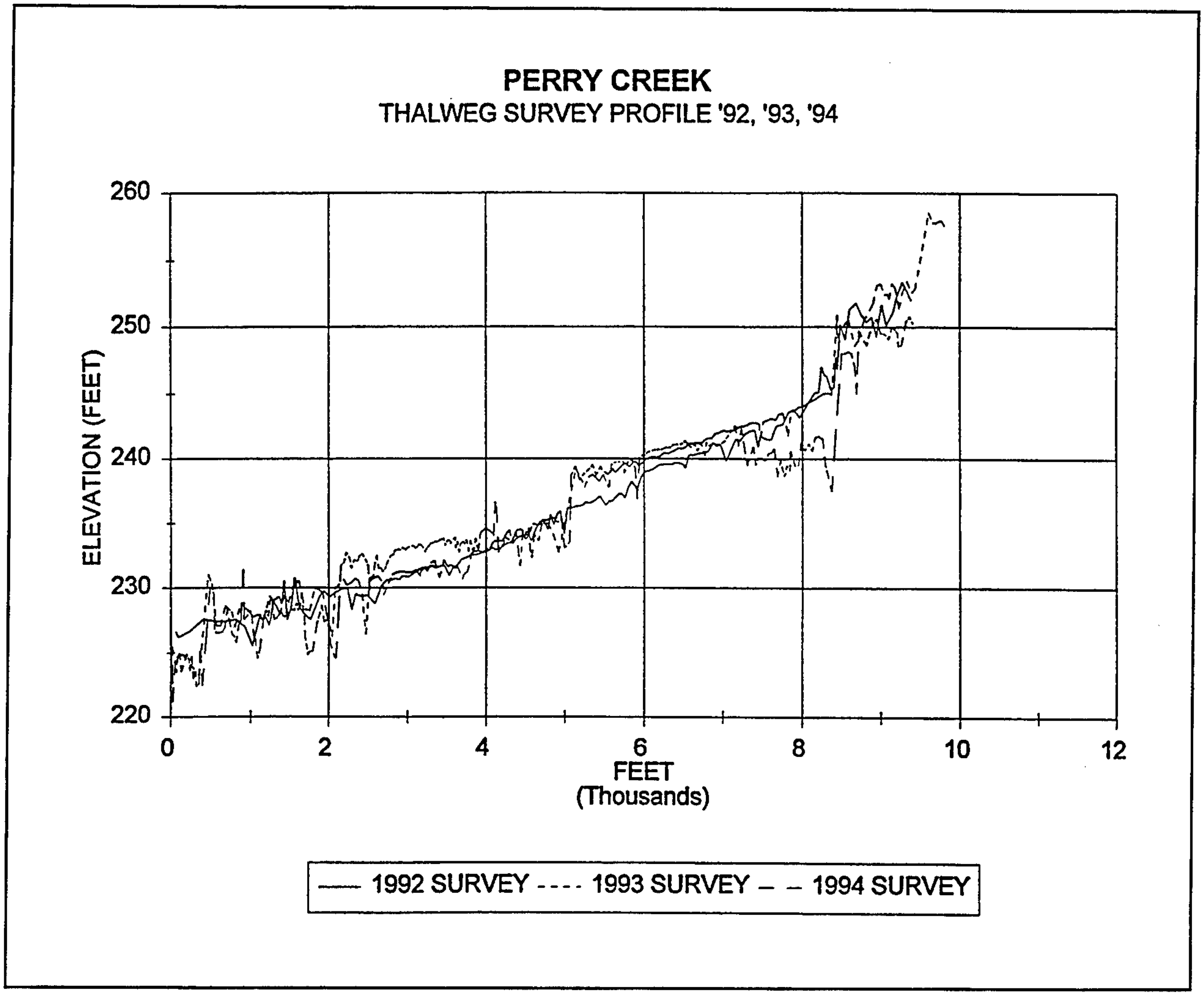

Figure 39. Perry Creek, Site 16, thalweg profile

Downstream channel degradation may have been set in motion by the initial construction of the channel stabilization structures and the narrowing at the bridge. Observed response upstream may have occurred more quickly because more frequent flows can move the fine sand required for aggradation; however, the downstream degradation was in clay and ironstone material requiring less frequent, greater magnitude events. In this scenario, decrease of the upstream supply caused by aggradation may have accelerated the response, but the controlling change in channel hydraulics would have been caused by the initial construction.

A more intensive investigation will be made of the conditions and circumstances occurring at the time of the chevron weir failure. At the present time, the data indicate a deepening of the local scour hole downstream of the downstream weir, hydraulic analysis indicates that the 2-year water surface slope had increased from 1992 to 1993, and the streambed downstream of the weir had degraded. Each of these factors would contribute to the potential to displace

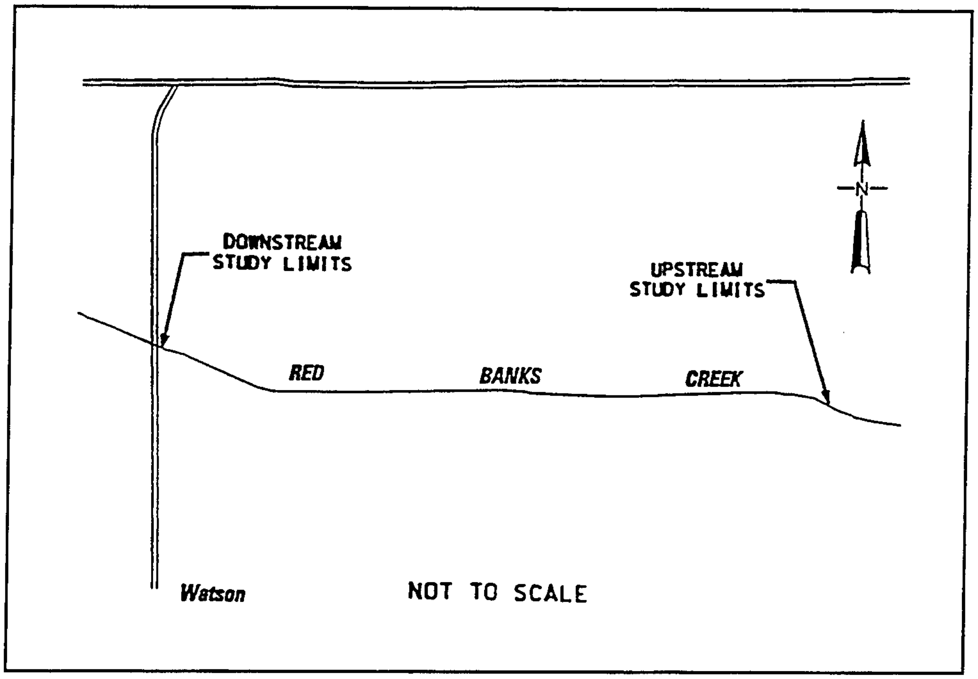

Figure 40. Red Banks Creek, Site 9

riprap in the chevron weir. It should be noted that in 1994, Shell Oil pipeline interests constructed a drop structure approximately 2,000 ft downstream of the study reach. Although at the time of the most recent field inspection, which was only a few months following construction, the amount of aggradation that will result from this structure could not be determined, backwater was observed up to the downstream extent of the study reach. See the section on Red Banks Creek in Chapter 6.

Sarter Creek, Site 15

Site 15 is on Sarter Creek, which is a tributary of Otoucalofa Creek upstream of Site 14. Sarter Creek is located on the Paris quadrangle map, T10S, R3W, Sections 34 and 35, and has a watershed area of approximately 6.4 square miles. The study reach is 4,000 ft in length and is almost completely straight as a result of previous channelization (Figure 42). This site extends downstream of the Highway 315 bridge. The site is unusual in that it has remained relatively unchanged since channelization; however, it is apparent that headcutting affected the reach in 1993 and continued to degrade the bed in 1994. Sarter Creek is degrading along the study reach as shown in the accompanying thalweg profile (Figure 43). Field inspection in November 1994 revealed headcuts near stations 18+00 and 9+00. Beaver dams control most of the upper 2,000 ft of the reach, and no headcuts were observed during the inspection.

BURBANK analyses indicate that the banks are not yet geotechnically unstable (Table 23). The SAM analyses indicated that the average width and

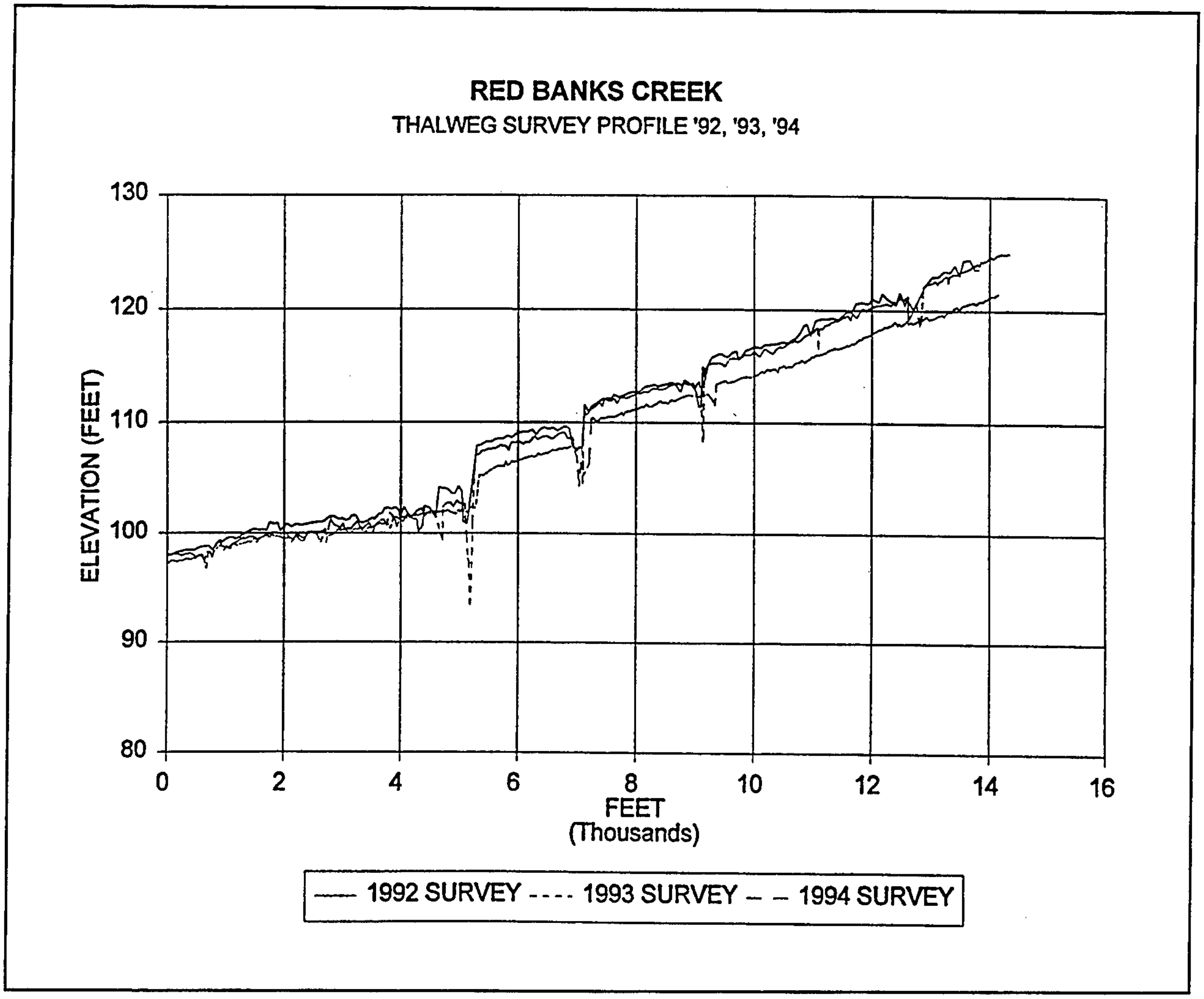

Figure 41. Red Banks Creek, Site 9, thalweg profile

slope are remaining constant at about 117 percent of the width and 170 percent of the slope required for minimum stream power for the transport of 1,000 mg/ℓ.

In general the bank stability remains good because the bank height is low, but the degradation potential is high. The channel is relatively small, only about 30 ft in width, and consideration should be given to testing lower cost grade control structures, such as gabion structures, to stabilize the channel.

Sykes Creek, Site 17

Site 17 is located on Sykes Creek as shown in Figure 44. The study reach extends 2,000 ft upstream and downstream of the county road bridge across Sykes Creek located in T21N, R5E, Sections 27, 33, and 34. This site is found

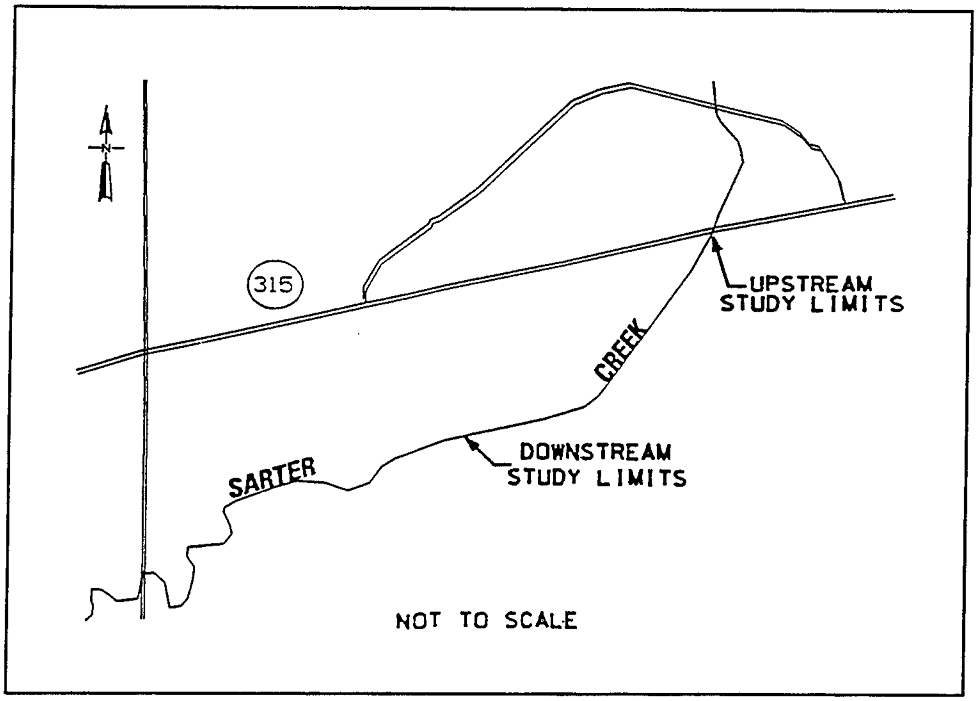

Figure 42. Sarter Creek, Site 15

on the McCarley quadrangle map. Gauging is presently available for the approximate 12.3-square-mile watershed area at the county road bridge.

The accompanying thalweg profile (Figure 45) indicates that little change has occurred during the 3-year period. Several factors relative to the site, such as berm formation, depth of sand in the bed, and thalweg comparison, indicate that the channel could be considered in quasi-equilibrium. However, comparison of the existing conditions with the slope and width required at minimum stream power for transport of 1,000 mg/ℓ indicates that the reach is transporting a relatively high sediment load. Based on the 1994, 2-year flow, the water surface slope is 235 percent of the minimum slope and the width is 70 percent of the width at minimum slope (Table 24). Also, the BURBANK analyses indicated that approximately 40 percent of the channel banks are unstable if degradation of 1 ft or more occurs.

About 300 ft upstream of the county road bridge a large, tight bend is active and in the process of producing a channel cutoff. During the present monitoring program the point bar chute has progressively enlarged, and the upstream approach has become more abrupt. At the same time the old channel has become increasingly choked by debris. Presently the upstream approach bend is eroding heavily into a residential lot, and, following the cutoff, the banks adjacent to a downstream hayfield will probably erode. The alignment to the downstream bridge is presently relatively straight, and the new alignment is uncertain. The cutoff will increase the slope locally and may cause upstream

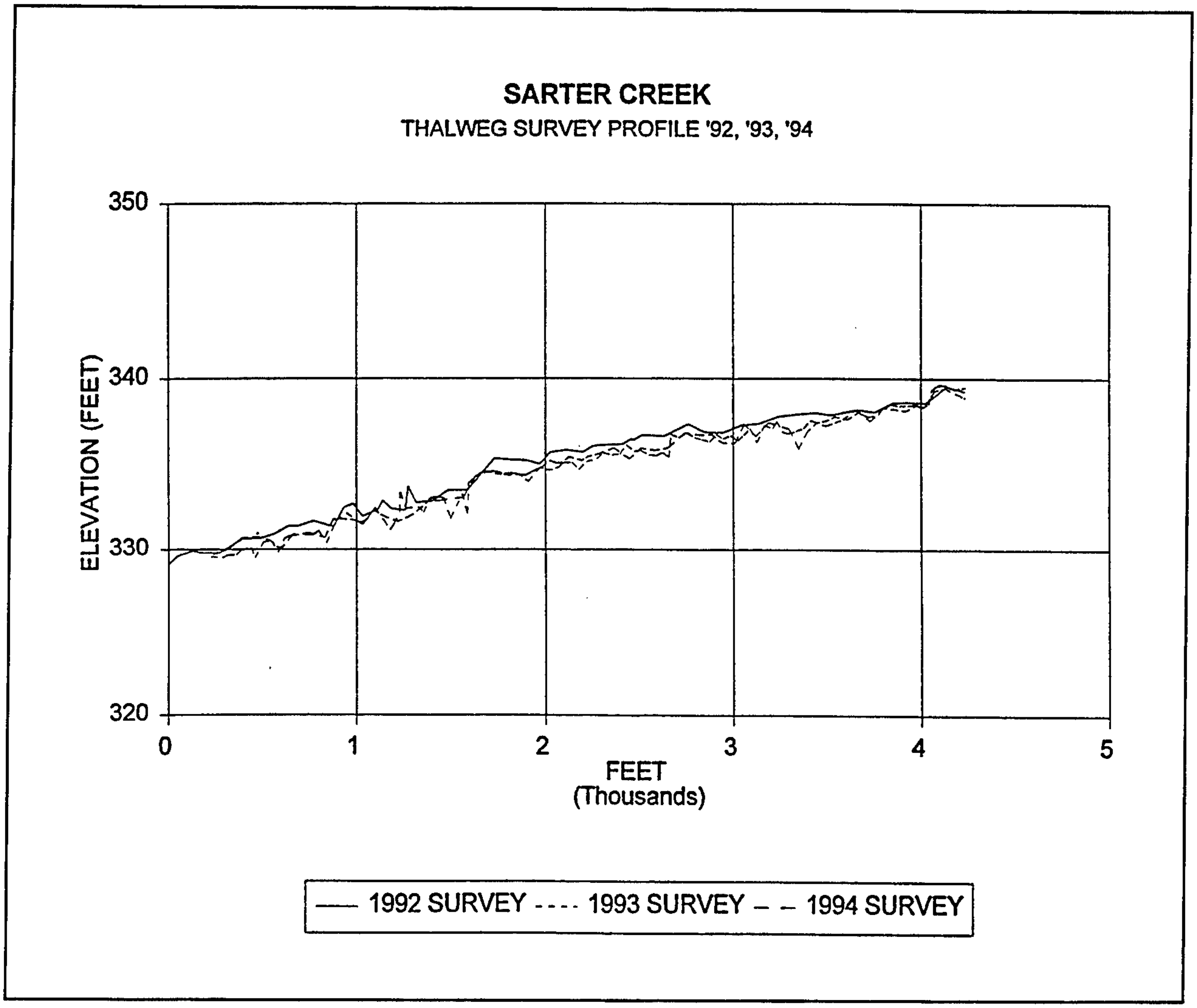

Figure 43. Sarter Creek, Site 15, thalweg profile

degradation. To fully evaluate the short- and long-term impacts of the changes occurring in this reach, consideration should be given to identification of upstream sediment sources, remedial measures to reduce sediment supply, possible future grade control, reduction of the slope and the bank height, and attention to the bridge alignment.

East Worsham Creek, Site 18a

Site 18 is a study reach encompassing portions of Worsham Creek, East Worsham Creek (Site 18a), West Worsham Creek (Site 18c), and Middle Worsham Creek (Site 18b), as shown in Figure 46. The site is located on the Duck Hill quadrangle map in T20N, R6E, Sections 14, 15, 16, 21, 22, and 23. The total stream length being surveyed is approximately 5.3 miles, and the watershed area at the confluence of Worsham and West Fork is approximately

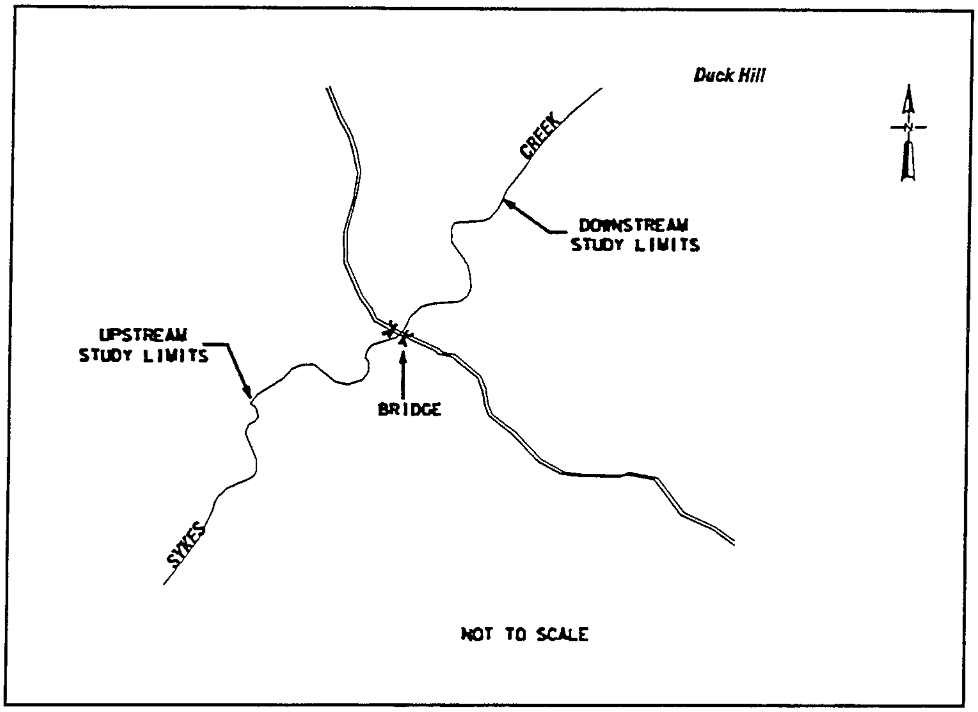

Figure 44. Sykes Creek, Site 17

19 square miles. The streams are deeply incised and active. Ten low-drop structures have been constructed in this study reach.

The downstream extent of the reach defined as East Worsham Creek (Site 18a) is the confluence of Worsham Creek with Middle Worsham, which is the first confluence downstream of the highway bridge. The thalweg profile indicates the locations of the structures and the variations in the profile indicating a general tendency for this site to degrade over the present monitoring program (Figure 47).

The reach is divided into three segments. Segment 1 extends upstream from the confluence to the downstream, older structure. The short reach between the two structures was not analyzed separately. Segment 3 is upstream of the middle structure. A beneficial effect of the upstream grade control structures can be realized by comparing the percentage of bank at risk for the zero friction angle condition in 1994 (Table 25). Grade control has raised the channel bed to reduce the percentage of unstable bank from 98 percent in segment 1 to 0 percent in segment 3. This is a significant improvement. However, the 2-year slope in both segments remains very high at approximately 300 percent of the 1000-mg/ℓ minimum slope. Fortunately, the segment 3 bed is composed primarily of very erosion resistant clay, and segment 1 includes numerous outcrops of ironstone and clay that provide a degree of vertical stability. Sediment transport in both segments is significantly controlled at lower discharges by beaver dams. The durability of the beaver

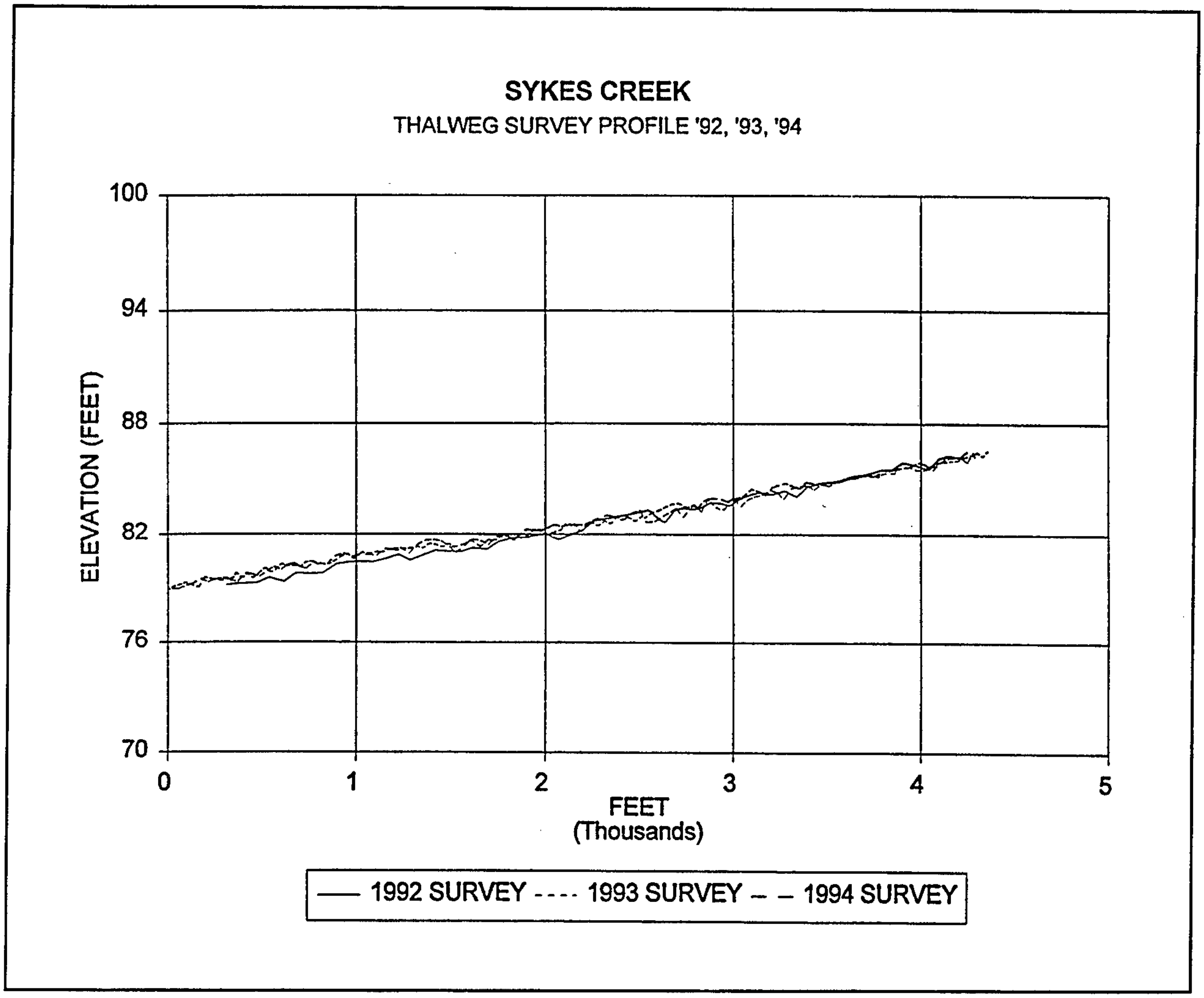

Figure 45. Sykes Creek, Site 17, thalweg profile

dams during high flows, time required to rebuild the dams, and amount of flushing of sediment through the beaver ponds is not known.

Middle Worsham Creek, Site 18b

The downstream extent of Middle Worsham is at the confluence of Worsham Creek with West Worsham Creek. The total reach is approximately 10,000 ft in length and is divided into four reaches by three ARS-type, low-drop structures. The accompanying thalweg profile (Figure 48) depicts the degradation that has occurred since 1992 in the lower two segments. Immediately upstream of the confluence of West and Middle Worsham, segment 1 has a shallow depth of sand, less than 2 ft, for most of the segment to the confluence with East Worsham. Upstream of this confluence, the channel is narrower and knick zones are present. Two drop pipes had been cleared for surveying in this reach. In segment 2, as the upstream structure is approached,

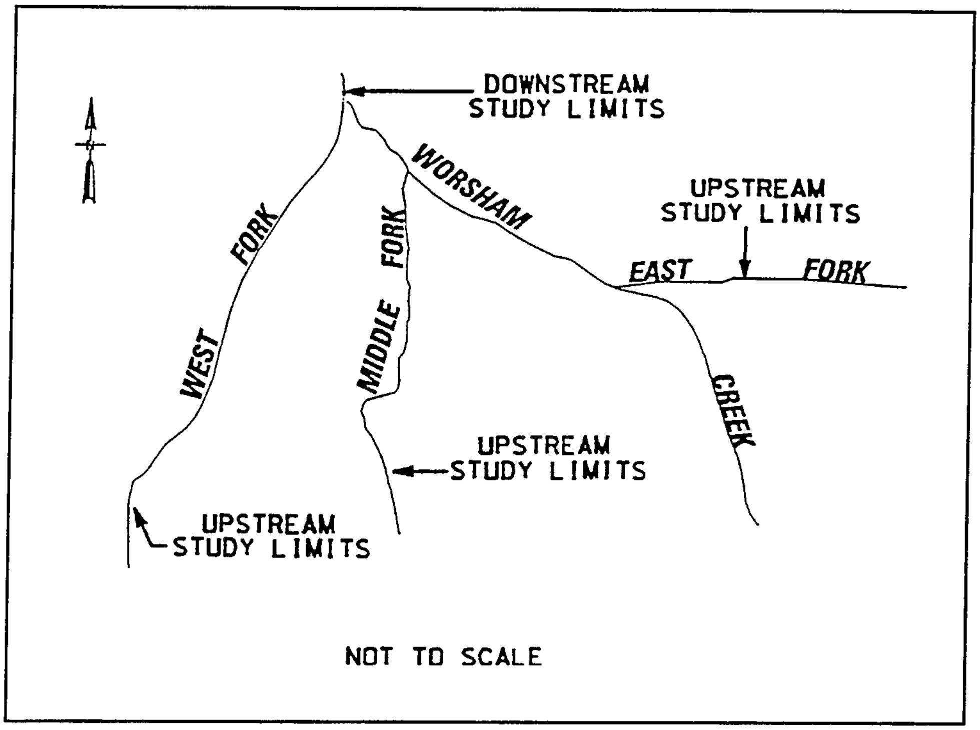

Figure 46. Worsham Creek, Site 18

knick zones and massive bank failures are present. The gully of the left bank downstream of the newer structure has been rehabilitated, and the structure has resulted in channel aggradation upstream to the channel ford, which is immediately downstream of the third structure. Headcutting upstream of this point is moving into the older structure slowly, and the upstream structure basin has significant sand deposits and willow growth. Upstream of the third structure, knickpoints are present at several locations. The right bank is eroding and consideration should be given to fencing the right bank upstream of the third structure to limit cattle access. Some form of grade control may be appropriate in the upper segment.

The BURBANK results clearly show the value of the grade control structures in reducing or maintaining low bank height (Table 26). Percentage of bank height unstable for the zero friction angle decreased from 100 percent in the lower reach to only 3 percent for the upstream reach. Since the lower reach has no grade control, the percentage of bank unstable has increased as the channel has degraded. With the relatively low slope in segment 2, filling may occur as the reach upstream of the third structure begins to pass sediment through the structure. Sediment has filled to the crest at this structure. Widths are narrow throughout the site.

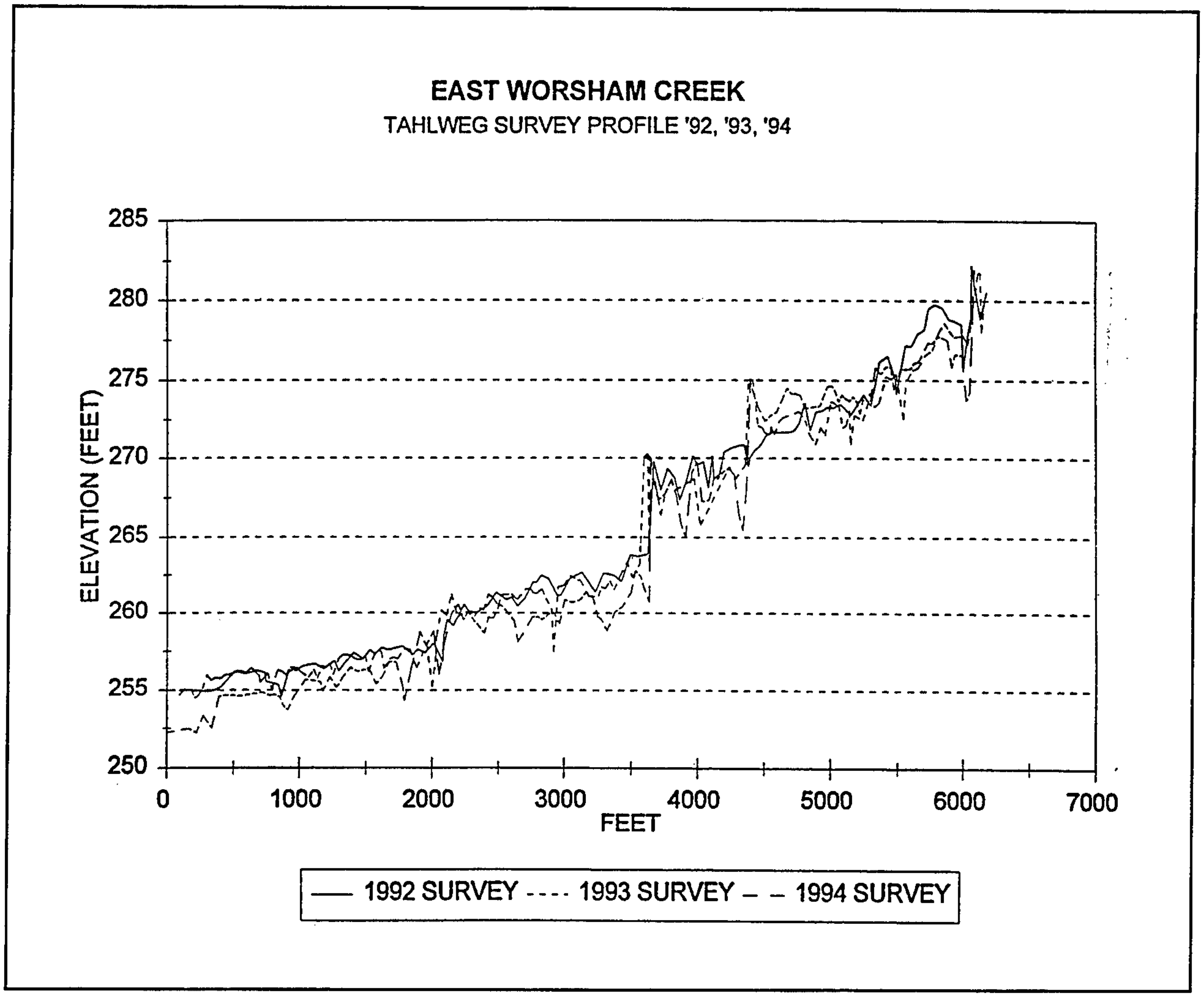

Figure 47. East Worsham Creek, Site 18a, thalweg profile

West Worsham Creek, Site 18c

The total reach length is approximately 10,000 ft and is divided into four segments by three ARS-type, low-drop structures. The thalweg profiles (Figure 49) indicate some degradation in the lower two segments, and clearly indicate that the aggradation has occurred immediately upstream of the newer structure. Note that the 1994 profile indicates filling to the weir crest of the newer, second structure, while the older, first structure has not filled. This indicates that the improved hydraulic control of the newer design results in improved performance, and suggests that renovation to improve hydraulic control is worth consideration. It should be noted that beaver dams are abundant immediately downstream of the first structure and upstream of the second structure. The gully into the second structure has been rehabilitated, and clearing has been accomplished for work at the downstream right bank gully within that site.

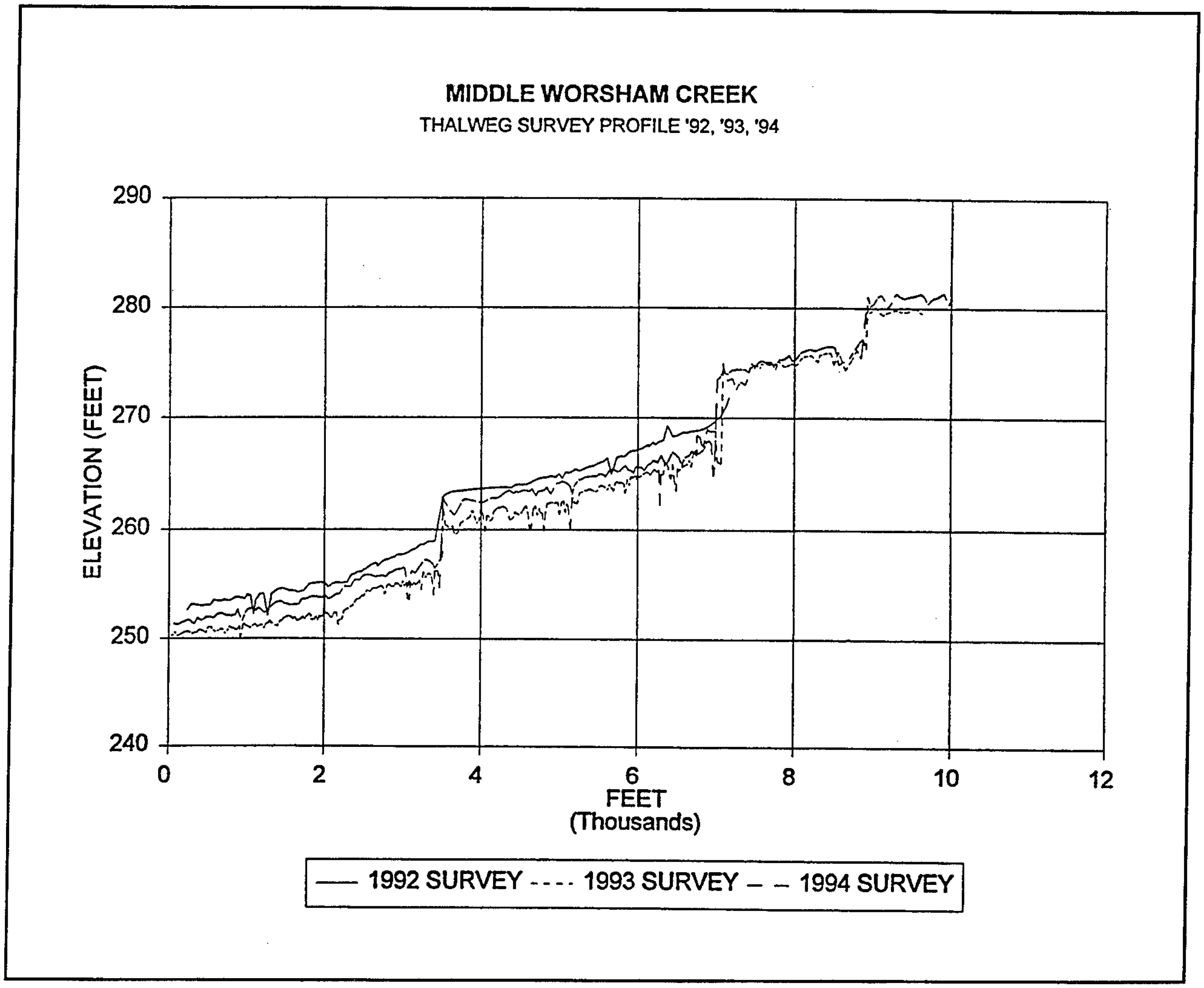

Figure 48. Middle Worsham Creek, Site 18b, thalweg profile

The BURBANK results indicate that banks are relatively stable, except in the lower segment (Table 27). This was confirmed by field inspection in October 1994. The 2-year water surface slope of the reach averages 315 percent, and the average stream width averages 63 percent of the slope and width required at minimum stream power for transport of 1,000 mg/ℓ. Although these data indicate a relatively unstable, incising channel, significant grade control has been placed in the system. The newer, second structure in segment 3 has been very effective in reducing slope, from 557 percent to 218 percent of the slope at minimum stream power for transport of 1,000 mg/ℓ. Consideration should be given to adding grade control downstream of the confluence of West and Middle Worsham Creeks.

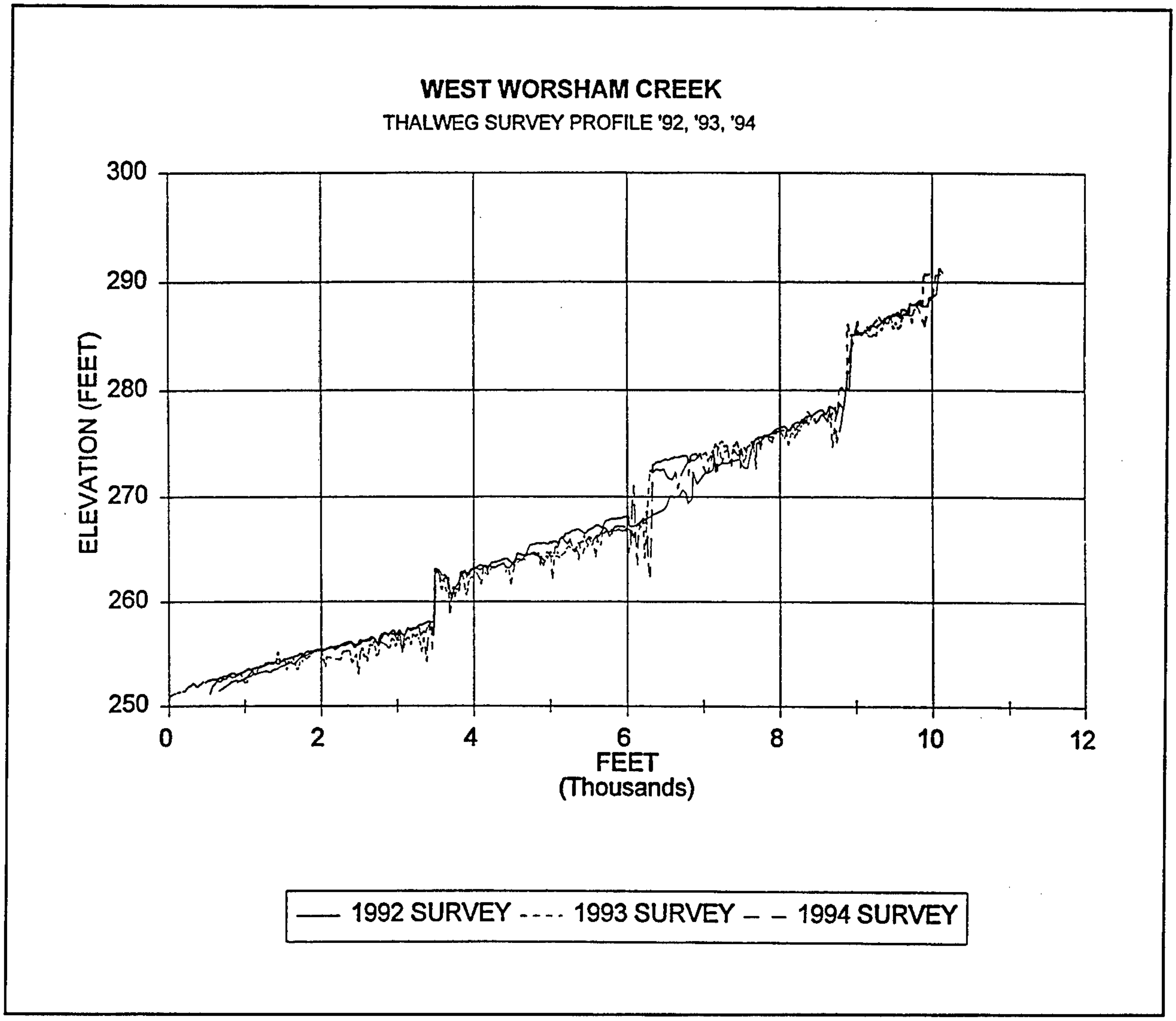

Figure 49. West Worsham Creek, Site 18c, thalweg profile

Sediment Reduction Capacity

In FY 1992 a procedure was developed to determine the sediment transport capacity of the 20 DEC sites being monitored at that time. This procedure was used as an evaluation tool to compute the sediment transport capacity for each site and for comparison of sites. For each of the 20 sites, the 2-year discharge was approximated using the method proposed by Colson and Hudson (1976) or the 2-year discharge provided by the Vicksburg District. In 1992, channel cross sections were surveyed at 500-ft intervals along each of the 20 study reaches. The survey data were used to generate a HEC-2 deck for each of the reaches. Repeated runs of HEC-2 over a range of discharges were used to determine the bank-full flow in each reach. The smaller value of either the 2-year discharge or the bank-full discharge was selected. A thalweg survey was

conducted during the summer of 1992. The thalweg profiles were then used to subdivide each reach into segments of relatively constant slope. For each segment of each reach the sediment transport capacity was computed using SAM. SAM uses the results of the HEC-2 run for each reach to compute the average hydraulic characteristics, effective width, depth, and slope for each segment of each reach. The sediment transport capacity is computed based on the average hydraulic characteristics. The total sediment transport capacity for each study reach was found by averaging the sediment transport capacity in the segments.

In FY 1993 and FY 1994, the same procedure was repeated. The total sediment transport capacity computed for FY 1993 and FY 1994 was compared to that computed for FY 1992. This comparison showed a 19 percent reduction in the total sediment transport capacity of the reaches being monitored between FY 1992 and FY 1994.

Summary

The performance of constructed features and the relative stability of the site are summarized as follows:

Abiaca Creek, Sites 3, 4, 6, and 21 and Coila Creek, Site 5. At the time of the October 1994 field inspection, no constructed features were in place. A sediment trap at Highway 49 is planned. The sites are generally stable, with minor local erosion and high sediment load.

Burney Branch, Site 12. Two high-drop structures are located in this site and provide an excellent example of grade control success.

Fannegusha Creek, Site 2. A low-drop structure was constructed in 1993, and preliminary indications are that the upstream bridge site has been successfully stabilized. Erosion appears to be continuing downstream, and some adjustment is continuing near the upstream extent of the site.

Harland Creek, Site 1. The longitudinal riprap is functioning successfully to stabilize the banks, and five bioengineering test sites have been proposed for future construction.

Harland Creek, Site 23. Willow posts and bendway weirs were installed as bank stabilization measures during late 1993 and early 1994. Scalloping between the bendway weirs is beginning to heal and filling between the weirs was observed. Willow post mortality is high; however, some bank stabilization has been noted. Investigations to resolve willow post mortality are underway.

Hickahala Creek, Site 11. Two low-drops are constructed on Hickahala Creek, and one about 700 ft upstream of the confluence on the South Fork tributary. All structures are functioning well; however, grouted rock at the left

bank weir of the downstream structure is cracking away from the weir cap and should be closely monitored. This structure is new and the stream continues to be unstable as the stream adjusts to control.

Hickahala Creek, Site 22. At Senatobia, the major channelization project was under construction at the time of the last field inspection in November 1994. Surveys will be made following construction as monitoring continues at this site.

Hotophia and Marcum Creeks, Site 13. Two new high-drop structures are located in the site, and an older, low-drop is at the downstream extent of the site. The high-drop structures are working well and sediment is depositing at the upstream structure. The downstream low-drop structure is holding several feet of grade on the downstream riprap and this should be closely watched. A drop pipe site immediately downstream of the Highway 315 bridge needs attention.

James Wolf, Site 19. The problems with the older, low-drop structure within the site have been previously discussed, and as of October 1994, no significant additional deterioration of the structure was noted. The structures continue to be successful in stabilizing the grade of the upstream segment of the site. Beaver dams have created backwater pools that have increased the effective height of the drop structure by about 2 ft.

Lee Creek, Site 10. No constructed features are located within the site. The landowner requested assistance at the site, and these problems are noted in the discussion. The downstream portion of the site is relatively stable, and the upstream portion of the site is somewhat laterally unstable due to island formation and erosion.

Lick Creek, Site 8. The channel is unstable and degrading throughout the site, and a new high-drop structure is in the final stages of construction immediately downstream of the bridge. Thalweg surveys during June 1995 should give a good indication of the initial effects of the structure.

Long Creek, Site 20. Five low-drop structures are in a series within approximately a 2-mile-long reach of the stream. Four structures are within the study site. Each of the structures appeared to be functioning properly during the October 1994 field inspection of the five structures. Longitudinal riprap is located on both banks of the channel for approximately the upper 5,000 ft. No significant rock movement was noted; however, some headcutting exists between structures in the reach. The reach should continue to adjust and is generally stable.

Nolehoe Creek, Site 7. The reach has no constructed features. Grade control was planned by the Vicksburg District, but a construction permit was denied by the landowner. The channel continues to incise and is unstable.

Otoucalofa Creek, Site 14. No grade control exists in the site. Bank stabilization placed along most of the reach is a combination of longitudinal riprap, toe dikes, and riprap blanket. A significant portion of these features have experienced stone movement and launching related to scalloping between dikes and channel incision. Grade control is planned for the site and the need for control is evident.

Perry Creek, Site 16. Four grade control structures have recently been constructed (1993) in the study reach, and indications are that this site is an excellent example of grade control success. Local erosion and the need for drop pipes are noted in the discussion.

Red Banks Creek, Site 9. Most of the banks within this reach have longitudinal riprap along both banks. Four chevron weirs were constructed in this site prior to 1992. Significant displacement of the downstream chevron weir occurred in late 1993, and the other chevron weirs were also damaged. Repair or replacement of the features should be considered.

Sarter Creek, Site 15. No constructed features exist on Sarter Creek, but since 1992 the reach has experienced headcutting and is in need of grade control.

Sykes Creek, Site 17. No constructed features exist on Sykes Creek. The channel is deeply incised, is meandering, and is transporting a high sand load. The bends are actively eroding. Generally, the reach has not significantly changed in the past 3 years. Consideration should be given to reducing the sediment load of the system.

Worsham Creek, Site 18. This site is at the junction of three tributary streams discussed in the text. The upper reaches of these streams are examples of grade control success, and because the lower reaches of these streams are not controlled by structures, the contrast between the upper and lower reaches clearly demonstrates the success.

The monitoring task, completed under contract with CSU, included two surveys of approximately 122,000 ft of stream channel in 1994. This included cross-section surveys in January and thalweg surveys in June. The 1994 surveys are the third data set for the DEC monitoring sites, and comparison of the 1992, 1993, and 1994 data has provided a basis for establishing trends in channel response and structure performance. For the period 1992 to 1994, the reduction in sediment yield at the 2-year discharge was approximately 19 percent.

Stable channel computations using SAM have been compared with composite cross-section data for each segment of the monitoring sites for 1992, 1993, and 1994. Comparison of the existing 2-year hydraulic gradient with the minimum slope channel morphology for the transport of 1,000 mg/ℓ has provided a baseline against which each segment can be compared.

The results of computations using BURBANK are presented for each site in tabular form. The analysis provided for the various monitoring sites indicated how BURBANK can be used to evaluate channel modifications such as grade control structures to quantify their success and as a design or monitoring tool. A combination of field survey and inspection, and computer analyses using HEC-2, SAM, and BURBANK provides suitable and comprehensive techniques for monitoring of the DEC Project.

4 Hydrology

Introduction

In the FY 1993 DEC report (Raphelt et al. 1995), a comparison of lumped and distributed models was presented. One of the lumped models used in that comparison was HEC-1, and the distributive model used was CASC2D. The principal objective of that study was to evaluate the watershed hydrology model, CASC2D, for application to ungauged watersheds. The simulation results showed that the CASC2D model will produce adequate results for design purposes with a limited amount of gauge data. Because a distributive model requires fewer subbasin stream gauge data than does a lumped model, it was recommended that the CASC2D model be used as an aid in the design and evaluation of streambank erosion and grade control structures in the future. In that report the following conclusions were also presented:

a. In the case where there is accurate spatial data representation of the watershed variability in soils and land use, a distributed model will simulate the true shape, rate of rise, and volume of the streamflow runoff hydrograph more closely than the lumped unit hydrograph methods.

b. In the case where sufficient subbasin stream gauge data are available for verification purposes, the lumped unit hydrograph models such as HEC-1 can reproduce the observed hydrograph reasonably well.

c. The lumped models rely heavily on subbasin stream gauge data to adequately simulate the observed hydrograph; however, CASC2D can simulate adequately as long as accurate spatial data are available. If accurate spatial data and subbasin stream gauge data are both lacking, then both models (i.e., lumped or distributed) may produce questionable results.

d. Since the distributive model CASC2D consistently produced more realistic results in terms of hydrograph shape and volume of runoff, it offers more flexibility when performing sediment studies than the lumped unit hydrograph models. This will be especially true when evaluating the

effects of specific land use changes or best agricultural management practices on erosion and sediment control within the watershed.

e. In this study, a GIS database had already been developed. Where a GIS database does not exist, a decision will have to be made as to whether an intensive stream gauging operation is more cost effective than developing data in a GIS. As time goes by, more GIS information will be available for a low cost. This should help to facilitate the development of a specific watershed GIS database and thus help to reduce the amount of stream gauge data needed. Once a GIS database is developed, a distributed model will be no more difficult to set up than a lumped model. In the event that a lumped model is still desired, the GIS data will help estimate the unit hydrograph and infiltration parameters with more accuracy than traditional methods.

It was recommended that the channel routing component of the CASC2D model be revised to more realistically represent the channel cross sections to improve the timing of the simulated runoff hydrographs. It was also recommended that the channel routing component be uncoupled or separated from the overbank routing component for modeling overbank flows. This would allow other numerical channel routing techniques to be evaluated and perhaps eliminate the stability problems caused by time-steps that are too long. For design of erosion control measures, the model must be able to handle high-intensity, short-duration storm events. It was also recommended that the CASC2D model be enhanced by adding sediment yield and transport subroutines for both the overland flow and channel routing components. This will allow evaluation of planned watershed best management practices and erosion or sediment control structures.

CASC2D Analysis

Introduction

In FY 1994 the WES effort focused on implementing the recommendations made in the FY 1993 report. To accomplish that, CASC2D analyses of some of the DEC watersheds were conducted. The CASC2D analysis consisted of developing the GIS data necessary for input into the model, modifying CASC2D for use in the DEC watersheds, and developing watershed models for Hotophia Creek, Hickahala-Senatobia Creek, Batupan Bogue, and Goodwin Creek.

GIS data

In developing the GIS data for use in CASC2D, the GRASS GIS was used primarily. The GIS data needed for use in CASC2D consisted of USGS DEM's, SCS soil texture maps, and land use maps. The soil texture data were

then reclassified into hydraulic conductivity, effective porosity, capillary head, and initial soil moisture maps. The land use data were reclassified into a Manning's roughness map, and the DEM's were edited to remove artificial pits and valleys and to create a seamless elevation grid. When the DEM's were patched together, seams were created, which needed to be removed for continuity.

Modifications to CASC2D

Due to numerical instability during low, base flow start-up conditions, the CASC2D model had to be modified to incorporate shaped cross-sectional data and Priessman Slots. The Holly-Priessman implicit channel routing scheme requires that a base flow be present at the beginning of each simulation; therefore, this channel routing scheme does not allow for dry channel bed assumptions as start-up conditions. Using a trapezoidal approximation of the measured cross sections resulted in channel bottom widths that were too wide. This caused the model to consistently compute supercritical flows and negative water depths. Adding the irregular shaped cross sections permits more realistic channel definition. Also, in some cases the channel slopes are too steep for the channel routing scheme to handle, which caused the model to consistently compute supercritical flows and negative depths again. To overcome this problem, the capability to input a Priessman Slot into each cross section was added and tested on the Goodwin Creek watershed data set. This addition allowed the slope to be reduced and smoothed without affecting the flood flows.

Hotophia Creek Watershed

The elevation grid for the Hotophia Creek watershed was edited, and overland flows were run to ensure that all artificial pits and valleys had been removed. Grids necessary for the infiltration routine were created, which included hydraulic conductivity, capillary head, effective porosity, and initial soil moisture. The channel network was developed using 1991 cross sections, and a roughness grid was created for use in computing overland flows. Finally, the initial water surface profile and the initial discharge profile were computed, and preliminary runs using the hypothetical rainfall were accomplished.

Hickahala-Senatobia Creek Watershed

The elevation grid for the Hickahala-Senatobia Creek watershed was edited and overland flows were run to make sure that all artificial pits and valleys were removed. All of the grids necessary for the infiltration routine were created including hydraulic conductivity, capillary head, effective porosity, and initial soil moisture. The channel network was developed using the 1991 channel cross sections, and a roughness grid was created for use in computing

overland flows. The initial water surface profiles and discharge profiles were computed. Preliminary runs were also conducted using the hypothetical rainfall.

Goodwin Creek Watershed

The work involving the Goodwin Creek watershed used a previously developed model. This watershed was used to help test and validate the modifications made to CASC2D presented in the preceding section. Since this is a small watershed, it was ideal for testing the modified routines in an efficient and timely manner. Two simulations were made using the new modifications. Improvements in computing the outflow hydrographs at various locations within the watershed were observed. Additionally, the modifications provided an increase in stability in computing initial conditions.

Batupan Bogue Creek Watershed

The elevation grid for Batupan Bogue Creek watershed was edited and overland flows were run to help eliminate artificial pits and valleys. In this case some additional editing was required before simulations could be conducted. All of the grids necessary for the infiltration routine were created including hydraulic conductivity, capillary head, effective porosity, and initial soil moisture. The channel network for this watershed was developed using the 1991 cross sections, and a roughness grid was created for use in computing overland flows. The initial water surface profiles and discharge profiles were developed. To complete the model on the Batupan Bogue watershed requires development of the initial channel conditions and the final elevation grid editing. Once those are completed, production runs can be initiated.

CASC2D Summary

The GIS data necessary for developing the remaining DEC watershed, which includes elevation, soil texture, and land use, have been developed. All that is necessary to set up the remaining watershed models is the development of the channel networks, incorporation of the channel cross sections, and hydraulic structures information. Then the elevation grids can be edited to remove pits and valleys. Some automation in developing the channel network and inputting the channel cross-section data has also been achieved. This should reduce the time required for development of future CASC2D models. Work will continue on developing the necessary GIS database on the remaining DEC watersheds for future sediment yield reduction studies. Improvements to the CASC2D model will continue as needed to accomplish these goals.

GISSRM Model Development and Testing

Introduction

In order to provide various alternatives of hydrology models for use in the DEC project, the GISSRM model was evaluated by the University of Memphis as one such alternative.

As discussed previously, WES conducted a study to compare the HEC-1 model to a new two-dimensional spatially distributed model CASC2D developed at CSU (Johnson 1994). In this analysis, all models used the Green-Ampt infiltration routine. The HEC-1 models used the Muskingum-Cunge channel routing routine while CASC2D used a one-dimensional diffusive wave routing routine. From the output, the peak flow, time to peak, volume of runoff, and hydrograph variance parameters were summarized for all three models.

The GISSRM model is very similar to the HEC-1 model using SCS unit hydrograph procedures and Muskingum-Cunge channel routing techniques. Therefore, this study was based upon the same subbasin and reach layout used in the 1994 study (Tables 28 and 29). However, additional watershed sediment data had to be collected and prepared for use with the GISSRM model.

Goodwin Creek watershed location and description

Goodwin Creek watershed has a drainage area of approximately 8.23 square miles and is located in Panola County in North Mississippi near Batesville (Figure 50). This watershed is a subbasin of the Long Creek watershed. The Goodwin Creek watershed was established to study the impact of land use and watershed processes on the stability of the channels and on the movement of sediment through the watershed. The watershed is highly instrumented with 17 rainfall gauges and 14 discharge and sediment gauges. This watershed has been extensively monitored by the ARS since 1981. Goodwin Creek watershed serves as a testing ground for sampling and predictive techniques of flow and sediment movement that will be used to evaluate erosion control and channel rectification measures on the DEC watersheds. The field sampling and measurement stations are located at channel grade control structures built in the late 1970's.

The land use within this watershed varies from forest (27 percent), to crop land (13 percent), to pasture (50 percent), and idle (10 percent). Over the past 10 to 20 years, this area has experienced streambank instability and sedimentation problems due to changes in land use. There are three primary soil types in the watershed: loam, sandy loam, and silt loam, with silt loam being the

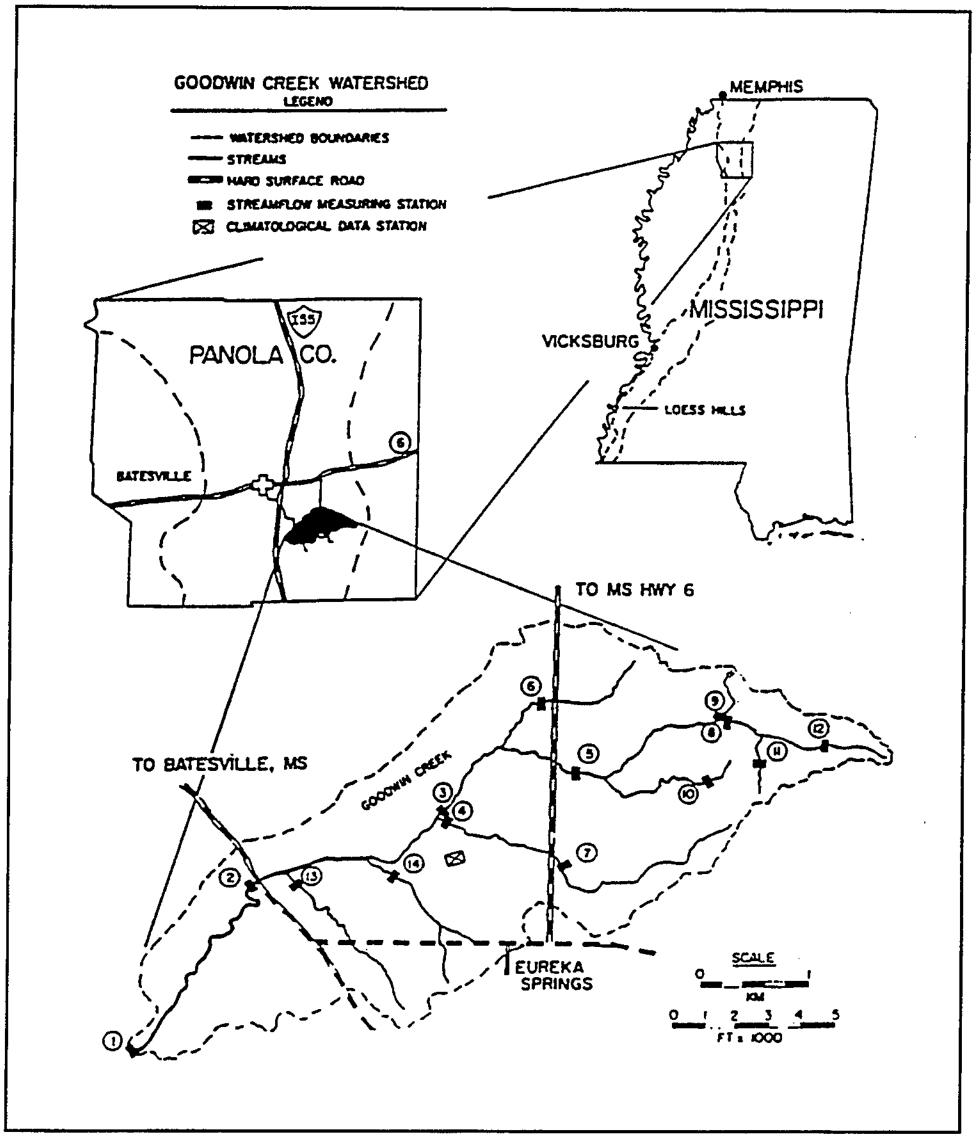

Figure 50. Location of Goodwin Creek watershed and the streamflow measuring stations

predominant soil type. Elevation[1] ranges from a maximum of about 413 to a minimum of about 237, with a mean channel slope of 21 ft per mile. The channels are so incised that out-of-bank flow rarely occurs. There is very little reported base flow between storm rainfall events for each flow measuring station. The normal annual rainfall is 55 in., as reported from the National Weather Service station at Batesville, MS.

[1] Elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

As part of the DEC Project, a GIS database using a square grid cell (416 ft) has been created by WES for Goodwin Creek. The GIS contains such data as land use grids, soil type grids, elevation grids, SCS curve number grids, slope grids, USGS digital line graphics, aerial photography information and other data. USGS 7.5-minute topographic maps covering the Goodwin Creek watershed include Batesville (1982) with a 10-ft contour interval and Sardis SE (1982), Courtland (1983), Shuford (1983), all having 20-ft contour intervals.

A computer listing of sediment sizes from a standard sieve or gradation analysis performed on 40 bed material samples was obtained from the ARS. These samples were taken in 1985 at 1,000-ft intervals. Sediment grain diameters D_{84}, D_{50}, and D_{16} along with the gradation coefficient were determined for each of these bed samples and plotted versus the distance or station above the outlet of the watershed. These plots (Figures 51 and 52) were used to determine an average of the sediment parameters over a routing reach used in the GISSRM model (Table 29).

Flow and sediment data over the period 1981-1992 were obtained from the ARS for the gauging stations. A regression analysis was made for four of the main channel gauges on Goodwin Creek. These gauges are shown on a channel thalweg profile (Figure 53) as Gauges 1, 3, 5 and 8. The gauge record data were analyzed to locate individual or single peaked storm events; and the peak flow, volume of runoff, and tons of sediment were computed for each storm event.

During a previous study by the ARS, a watershed sediment budget for Goodwin Creek was estimated. The budget (Table 30) was based upon estimated fines loading from documented land use and gauged fines loads per subwatershed over a 59-month measurement period from November 1982 to October 1987. Total runoff was 102.8 in. over this period. About 75 percent of the fines load and 85 percent of the total load are estimated to have originated from channel and gully erosion within the watershed. The annual yield for fines calculated from documented land use was based upon the following values: forest lands 0.12 ton/acre; pasture lands 0.4 ton/acre; cultivated farming lands 4.7 tons/acre; and idle lands 1.4 tons/acre. Even if the value for cultivated land was in error and should have been doubled to 9.4 tons/acre, the percentage of sediment yield due to gully and sediment erosion would be reduced only by 10 percent. Upland gully erosion could not be further separated from the channel erosion components. Based upon these last two statements and the values shown in Table 30, it was assumed for modeling purposes with GISSRM, to use a 30 percent to 70 percent ratio breakdown of the total measured load; that is, 30 percent for wash load from the subbasins and 70 percent for the channel bed material load. The total tonnage for each storm event summarized from the ARS records was then reduced by a factor of 0.30 before the study was conducted.

The universal soil loss equation parameters were used for each subbasin area, and the MUSLE equation constant and exponent were computed from the least squares regression analysis for each of the four gauge locations selected.

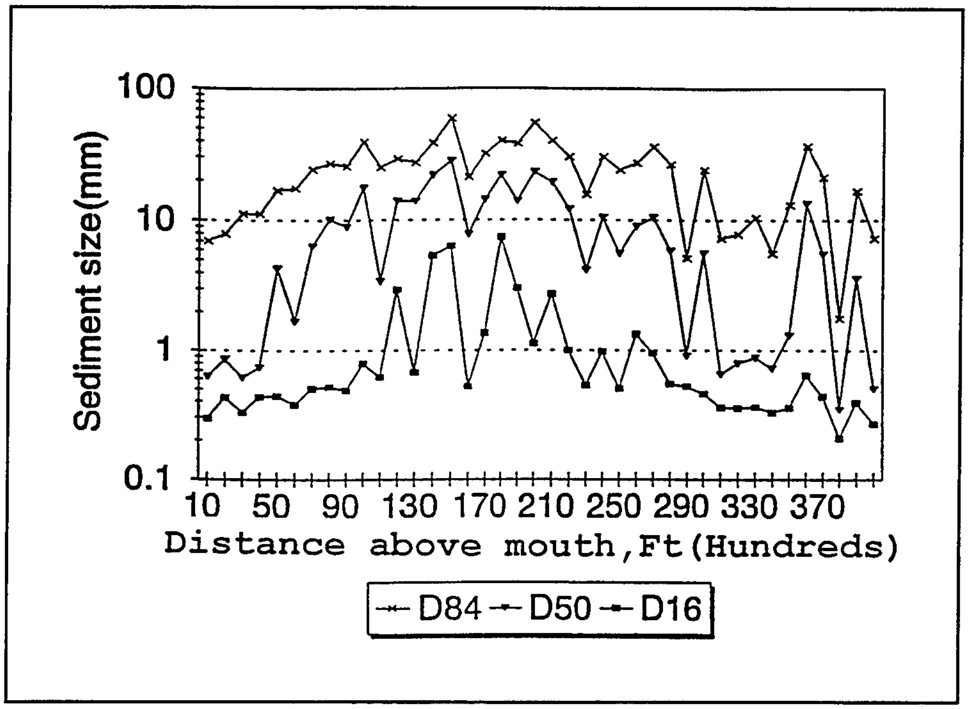

Figure 51. Goodwin Creek, sediment size along watershed

These data were plotted versus the drainage area contributing to the gauge, which allowed the development of equations for the MUSLE coefficients. The resulting regression summary statistics for these equations are shown in the following tabulation. The subbasin computed MUSLE coefficients used in the GISSRM model are shown in Table 31.

Regression Output for C1 versus Area:		Regression Output For C2 versus Area:	
Constant	1.044959	Constant	0.551823
Std Err of Y Est	0.243959	Std Err of Y Est	0.024833
R Squared	0.706464	R Squared	0.93383
No. of Observations	4	No. of Observations	4
Degrees of Freedom	2	Degrees of Freedom	2
X Coefficient(s)	-0.62285	X Coefficient(s)	0.02254
Std Err of Coef.	0.283895	Std Err of Coef.	0.004243

The same five storm events used in the WES comparison of HEC-1 and CASC2D were simulated using the GISSRM model. The current version of GISSRM uses the SCS procedures in a lumped storm subbasin hydrograph approach while the HEC-1 model computes runoff using a subbasin unit hydrograph approach and the CASC2D model is a spatially distributed (i.e., square grid cells) approach to overland flow. A comparison of peak flows for these five storms computed by all three computer models is shown in Table 32. A comparison of observed versus predicted sediment yields is shown in Table 33. Simulated hydrographs for the watershed outlet (Gauge 1) are shown in Figures 54 to 58.

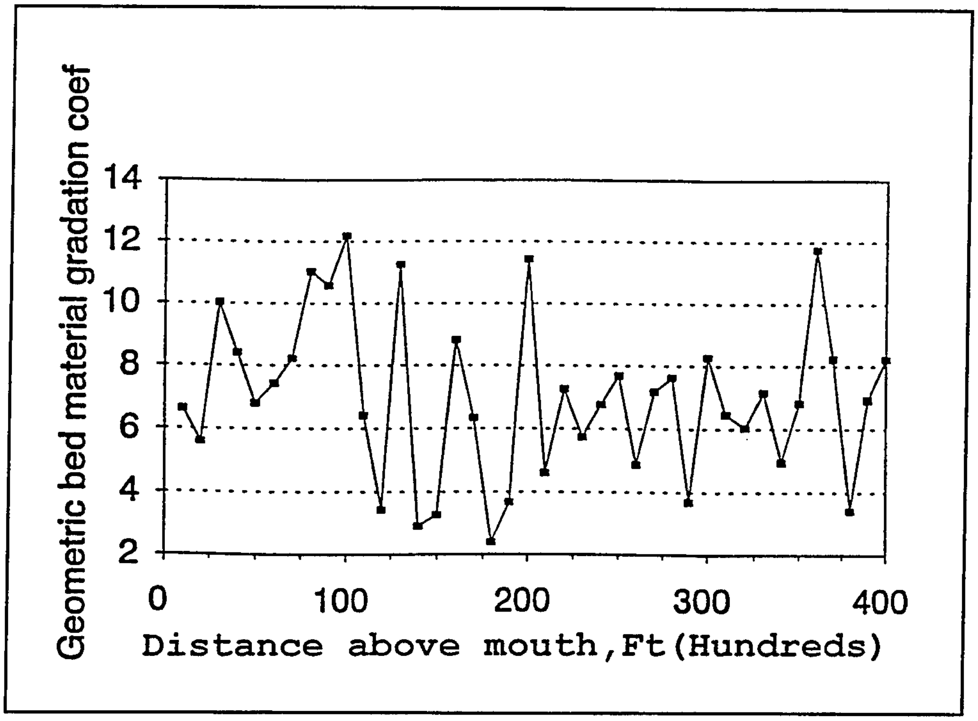

Figure 52. Goodwin Creek, bed material gradation coefficient

Summary of test results

Based upon a comparison with observed data for the historical storm runs, as well as the annual yield prediction, the synthetic routing methodology for flow and sediment used in the GISSRM model is giving reasonable results. The lumped storm hydrograph approach appears to give reasonable peak flow values (±25 percent) for uniform rainfall events when the storm duration is equal to or less than the watershed travel time. Storm sediment yields predicted were also within the expected range of accuracy (±30 percent). Therefore, in the future the GISSRM has potential for application to other DEC watersheds to determine the effectiveness of the grade control structures in reducing sediment transport.

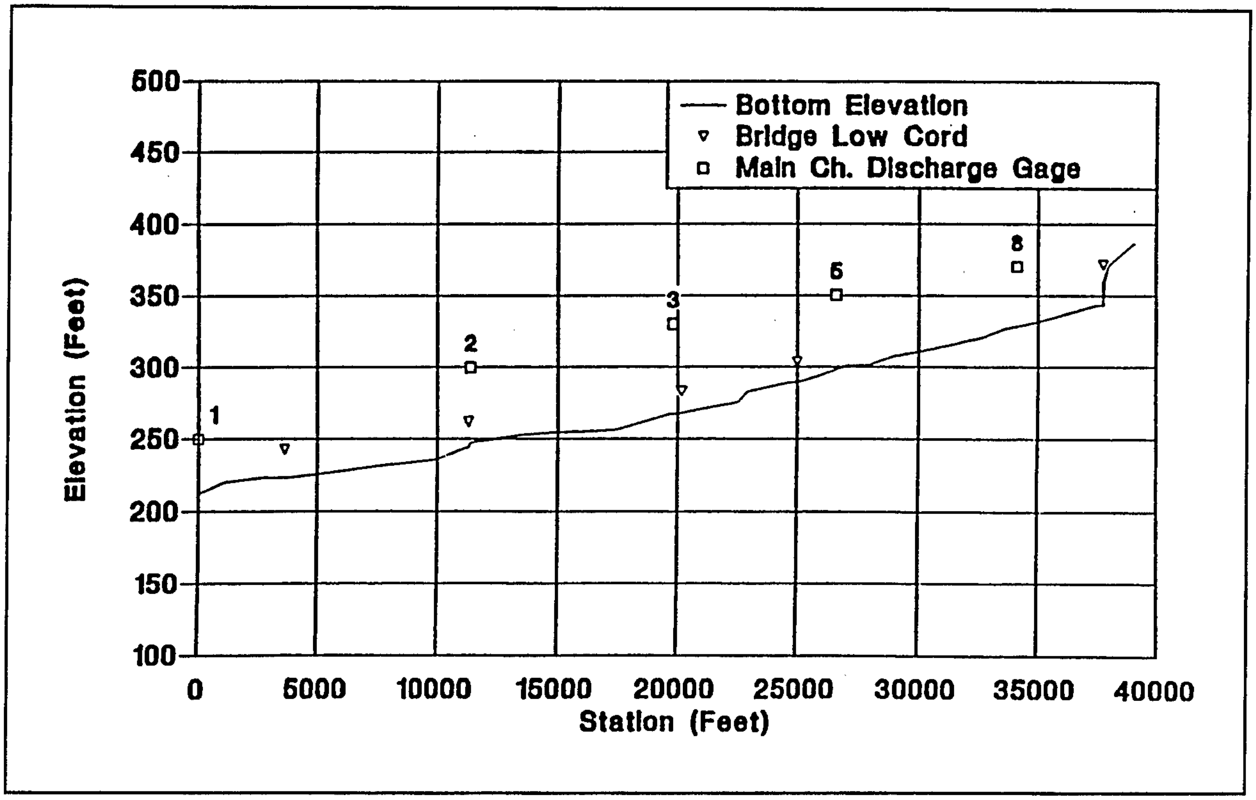

Figure 53. Main channel bottom profile, Goodwin Creek

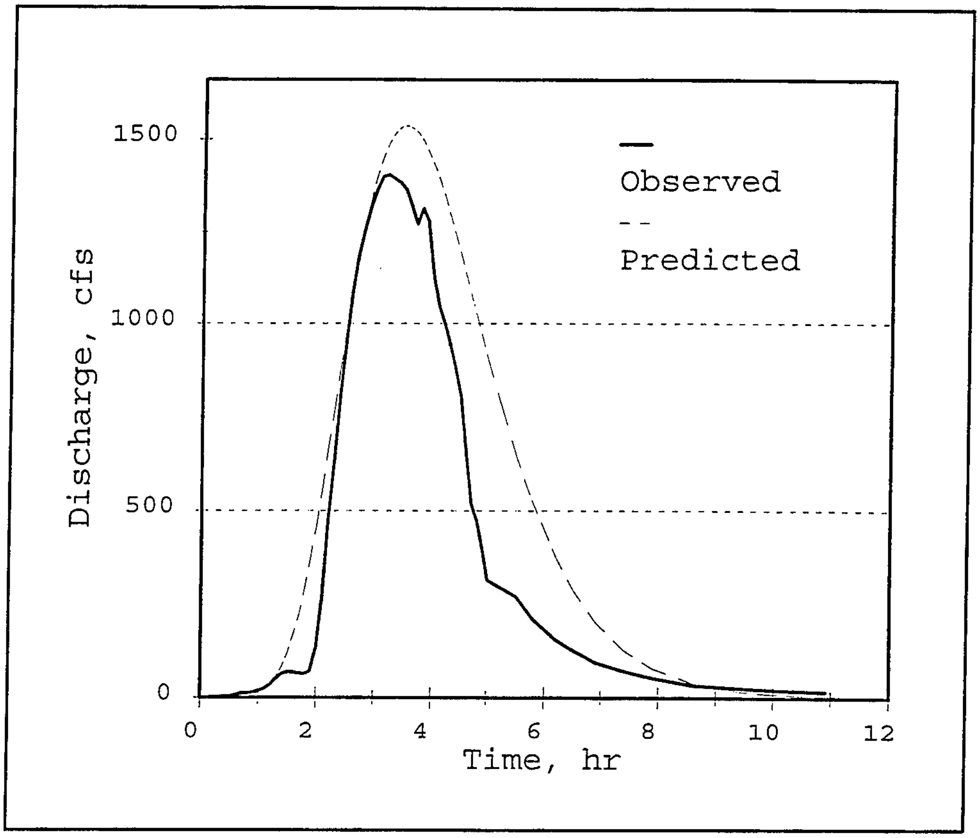

Figure 54. Hydrograph simulation comparison for storm event 1 (10-17-81), Goodwin Creek watershed (Gauge 1 at outlet)

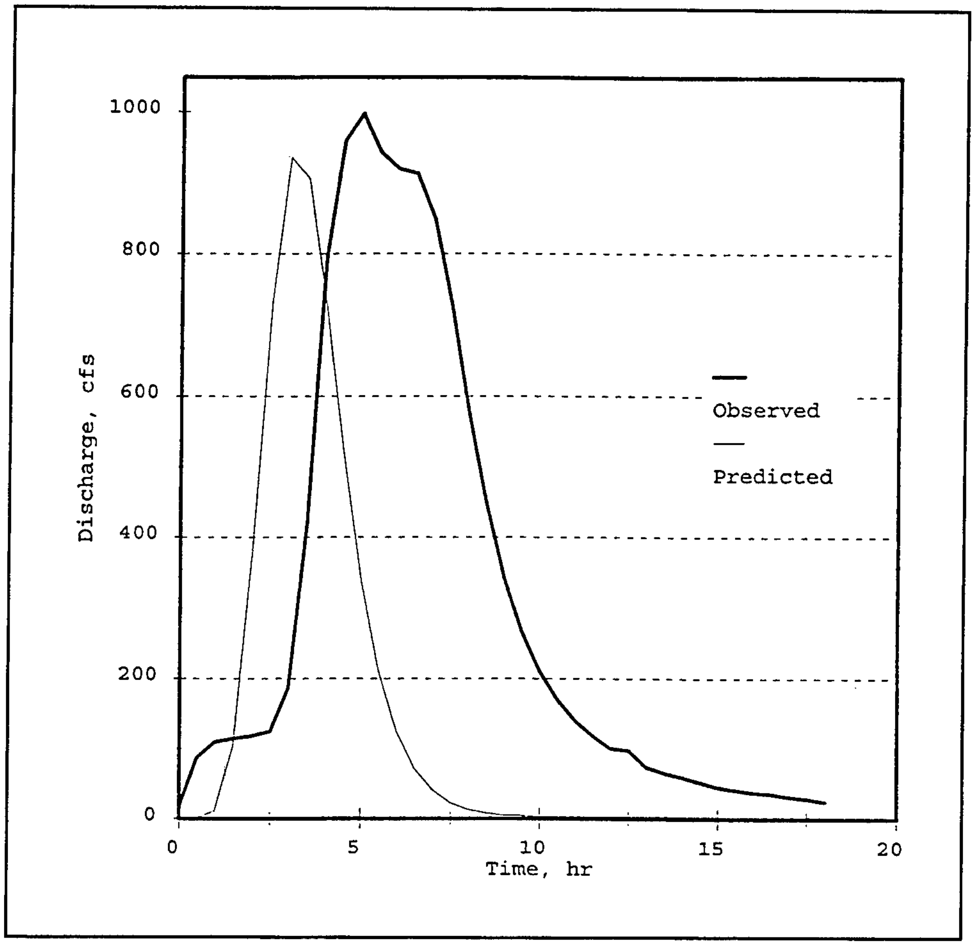

Figure 55. Hydrograph simulation comparsion for storm event 2, Goodwin Creek watershed (Gauge 1 at outlet)

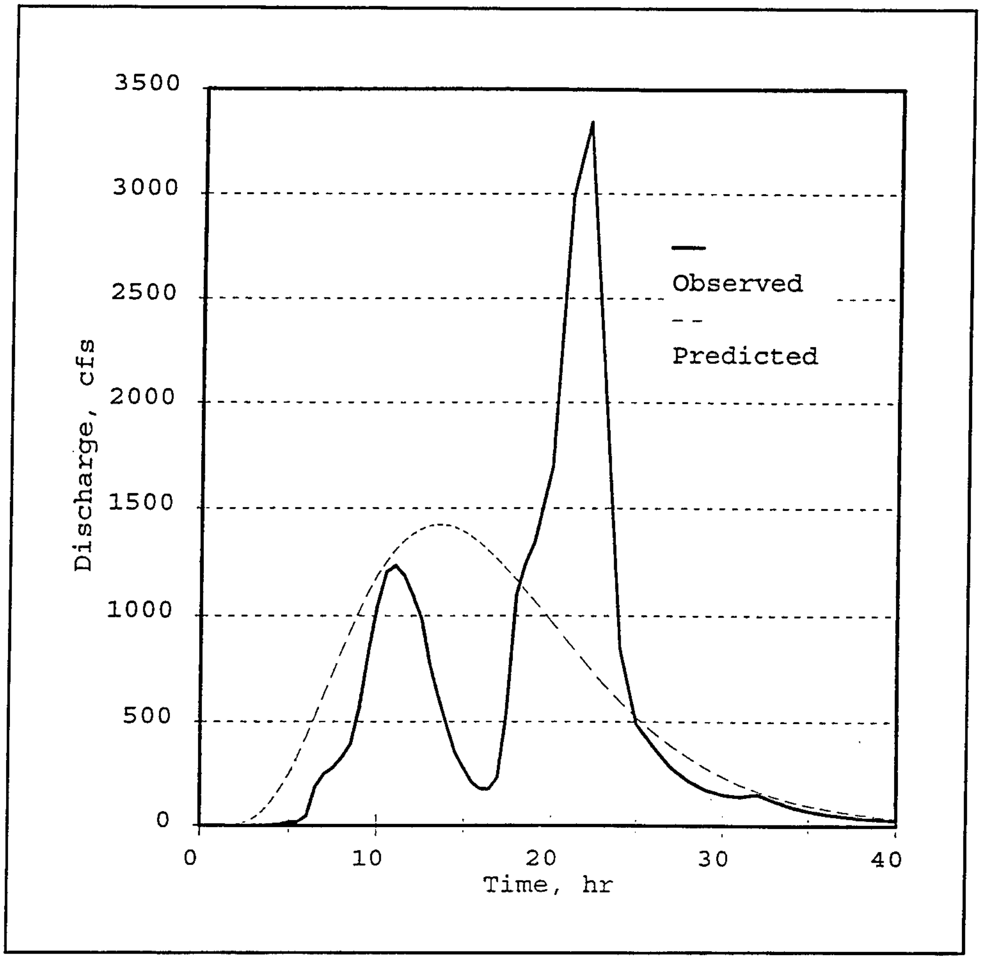

Figure 56. Hydrograph simulation comparison for storm event 3, Goodwin Creek watershed (Gauge 1 at outlet)

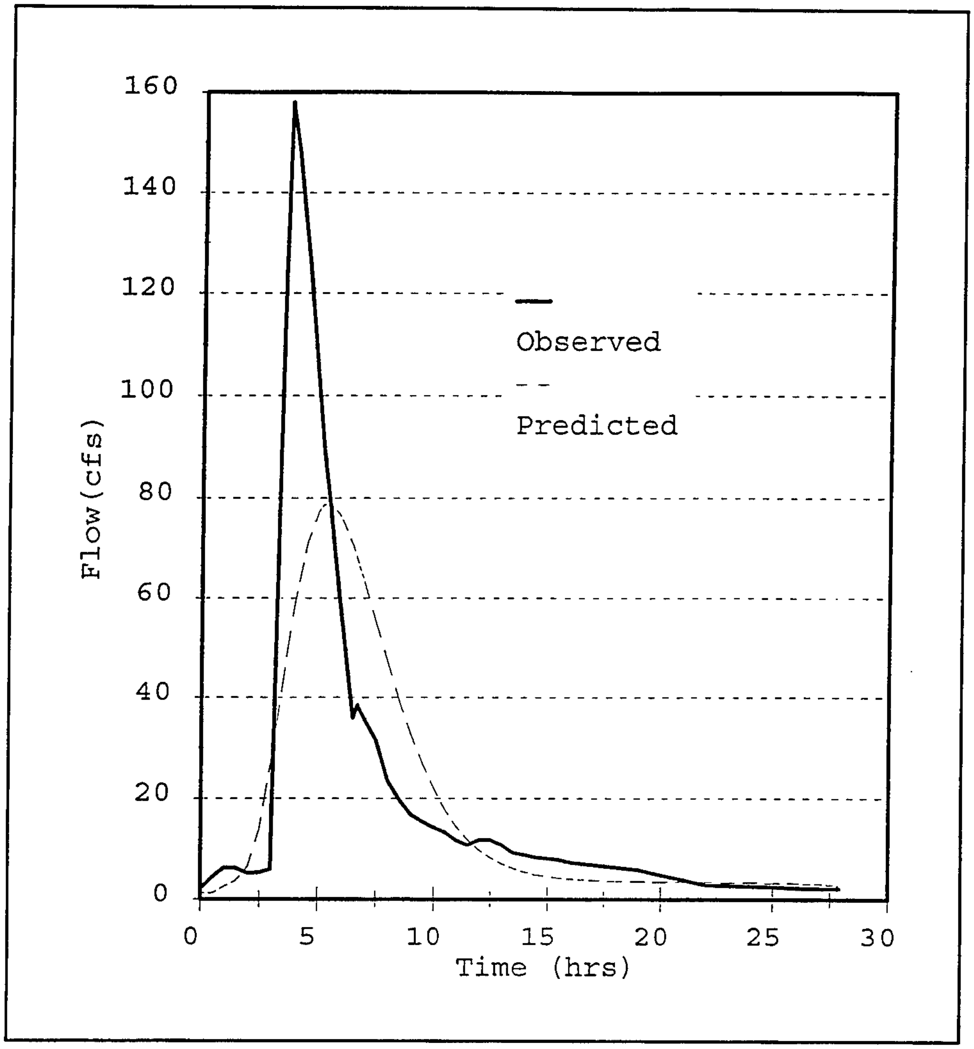

Figure 57. Hydrograph simulation comparison for storm event 4, Goodwin Creek watershed (Gauge 1 at outlet)

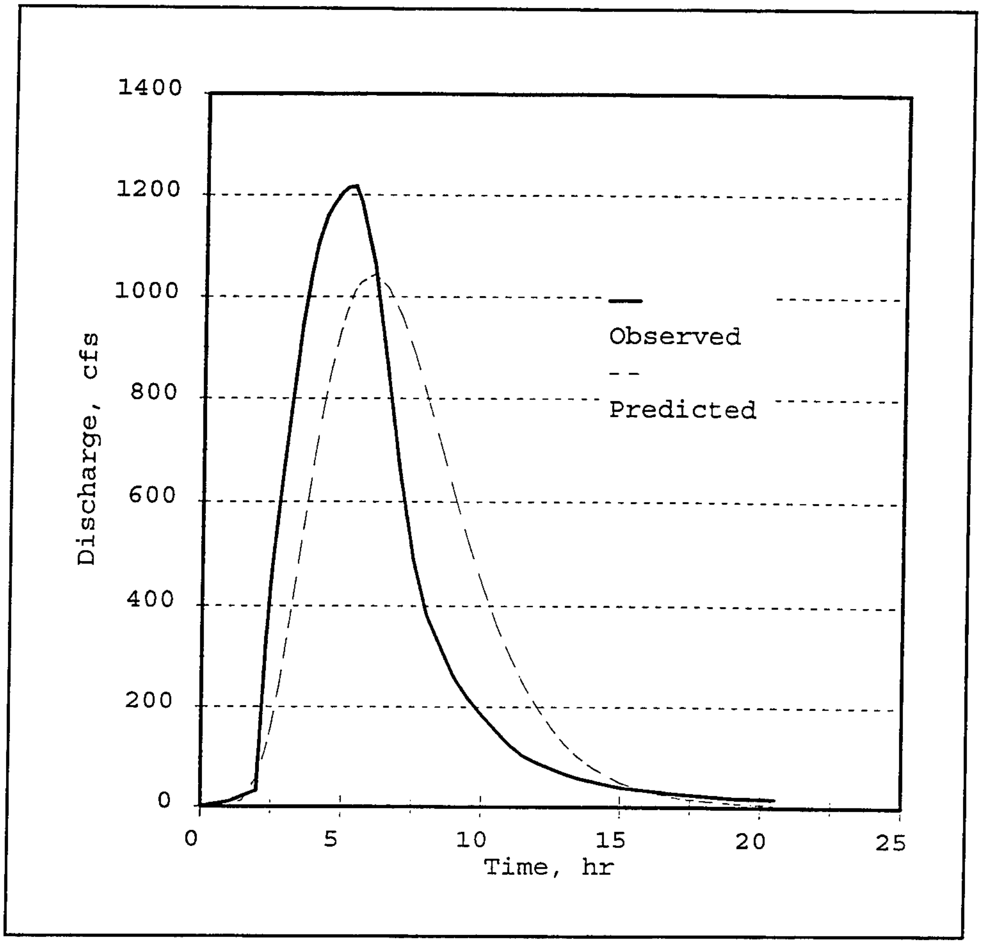

Figure 58. Hydrograph simulation comparison for storm event 5 (12-27-88), Goodwin Creek watershed (Gauge 1 at outlet)

5 Performance of Hydraulic Structures

Introduction

In FY 1994 there was no inspection of DEC low- and high-drop grade control structures. It was decided to establish an alternating sequence with inspection of the grade control structures during odd-numbered years and inspection of bank stabilization structures on even-numbered years. Therefore, other than the grade control structures included in some of the 23 individual monitoring sites presented in Chapter 3, no other DEC grade control structures were inspected during FY 1994. However, for this report to present a comprehensive state-of-the-DEC condition, a portion of the FY 1993 DEC report (Raphelt et al. 1995) has been included. The inspection of DEC low- and high-drop structures was conducted in FY 1995 by CSU. That inspection report will be included in the FY 1995 report.

FY 1994 Monitoring Sites

As presented in Chapter 3, the DEC monitoring sites inspected in FY 1994 with hydraulic structures in place are Burney Branch, Site 12; Fannegusha Creek, Site 2; Hickahala Creek, Site 11; Hotophia and Marcum Creeks, Site 13; James Wolf, Site 19; Lick Creek, Site 8; Long Creek, Site 20; Perry Creek, Site 16; Red Banks Creek, Site 9 (chevron weirs); and Worsham Creek, Site 18. Of the sites monitored in FY 1994, Hickahala Creek, Site 11, and James Wolf, Site 19, had hydraulic structures identified as possibly requiring maintenance or repair.

FY 1993 Inspection

Field inspections were made in June 1993 of the low- and high-drop grade control structures in the DEC project by CSU. A structure evaluation form for each structure was prepared and is presented in Watson, Abt, and Hogan

(1993). The report contains a series of photographic slides of each structure and transcriptions of approximately 5 hours of narrated and annotated video tape showing the 1993 field conditions. The inspection report is summarized in this section for convenience.

The six most common problems observed are as follows. The percentage shown for each problem denotes the percentage of total structures in which the problem occurred:

a. Riprap is displaced from the face of the weir (41 percent).

b. The channel bank upstream or downstream of the structure is failing (37 percent).

c. Bank erosion or piping beneath the riprap is occurring caused by overbank drainage (24 percent).

d. Riprap is launched at the upstream or downstream apron (28 percent).

e. Severe headcutting is migrating into the basin (17 percent).

f. Woody vegetation has become established in the upstream or downstream apron and is impairing the conveyance or the weir unit discharge of the structure (19 percent).

In addition to identifying the types of problems and making recommendations for resolving the problems, CSU assigned a priority to each structure as follows:

a. **Category 1** structures are under an imminent threat of loss of function.

b. **Category 2** structures have problems that should be resolved.

c. **Category 3** structures have no significant problems.

Table 34 summarizes the types of problems (problem types correspond to subparagraphs *a* through *f*) and the category for each structure evaluated. Four structures are in category 1, 32 in category 2, and 19 in category 3.

Summary

Of the 55 grade control structures inspected in FY 1993, 7 percent of them are in category 1, 58 percent are in category 2, and 35 percent are in category 3. The four category 1 structures in imminent threat of loss of function are all low-drop structures such as LD-1 on Deer Creek, LD-1 on James Wolf Creek, LD-1 on Little Bogue Creek, and LD-4 on West Fork of Worsham Creek.

6 Bank Stability

Introduction

Bank stabilization continues to be one of the focal points within the DEC Project. This area has been addressed by evaluation of existing banks and stabilization works, development of alternatives and/or modifications to typical riprap bank stabilization such as bendway weirs and willow posts, and development of additional bioengineering applications for the DEC streams. This chapter will address those issues based on work and evaluations performed during FY 1994.

Aerial Inspection

The purpose of the aerial inspection task is to identify, from aerial reconnaissance, the channels in the various watersheds that appear to be the most active with regard to bed and bank stability. The ARS Sedimentation Laboratory in a cooperative agreement with WES assumed the responsibility for obtaining aerial videos of the watersheds. The ARS uses Super VHS video equipment that records frames in digital format that can be easily read into the computer database. During the inspection, aerial videos were made on the main channel and major tributaries in each watershed from a fixed-wing aircraft flying at an altitude of 2,400 ft above the ground surface. A second flight was made over the 22 long-term monitoring sites at the same altitude but with the camera lens set to maximum magnification to get better resolution on the pictures.

Due to logistical problems and other factors, the aerial inspection was not conducted in FY 1994. Therefore, the last aerial inspection was conducted in the spring of 1993. See Raphelt et al. (1995) for a description and details relative to the FY 1993 aerial inspection.

Monitoring of Bank Stabilization

Introduction

In FY 1994 several existing DEC bank stabilization works were monitored by CSU. The first portion of the report submitted by CSU documenting this effort provides an excellent background on bendway migration fundamentals, bank stability and stream width adjustment mechanics, transverse dikes and bendway weir literature, and bioengineering principles (Watson, Gessler, and Abt 1995). The overall purpose of the research was to analyze the performance of bank stabilization techniques applied to arrest bend migration, to assess placement of riprap bank stabilization, and to monitor bank stabilization methods for the purpose of developing design guidance.

The methodology for this research combined field data acquisition, literature review, and review of the Vicksburg District channel stabilization plans and specifications. Field data were collected at the following Yazoo Basin sites:

a. *Goodwin Creek.* Longitudinal and transverse dikes downstream of the ARS measuring site for a distance of approximately 1 mile.

b. *Little Bogue Creek.* Riprap along both banks for approximately 1 mile upstream and downstream of the low-drop grade control structure.

c. *Otoucalofa Creek.* Riprap longitudinal and transverse dikes for a distance of approximately 2,000 ft upstream and downstream of the Mt. Liberty Church road bridge.

d. *Red Banks Creek.* Riprap longitudinal and transverse dikes for a distance of approximately 1 mile upstream of the Watson-to-Moscow highway in conjunction with v-notch weirs constructed of riprap.

e. *Harland Creek.* Experimental application of bendway weirs and willow posts downstream of Moccasin Creek for approximately 2 miles.

Field data included photographs and observations of launching, riprap displacement or failure, sediment deposition, vegetation, and general structural performance. Historical aerial photography was provided by the Vicksburg District. For those locations not obstructed by vegetation and for which photography was available, bank line comparisons were developed. In addition to bank line comparisons, construction drawings and surveys, available information pertaining to radius of curvature, channel width, channel approach alignment, bank materials and vegetation, and channel stability were collected and analyzed. Personnel from the Vicksburg District who were familiar with the site design were interviewed to ascertain conditions existing at the time of design and factors that may have influenced design and construction.

Goodwin Creek, Little Bogue Creek, Otoucalofa Creek, Red Banks Creek, and Harland Creek site evaluations are included in this section. Harland Creek is unique because it was stabilized using the experimental technology of combining bendway weirs and willow posts in an incised, meandering channel. The remaining four sites use more conventional methods of low-drop grade control and longitudinal and transverse riprap dikes. Although Harland Creek has been continually monitored by WES since installation of the bank stabilization works, the CSU monitoring provided an excellent opportunity to obtain a second evaluation of the bendway weirs and willow posts on Harland Creek. The WES evaluation is presented later in this chapter.

Stabilization techniques evaluated

As presented in the previous section, the techniques evaluated on the various DEC streams included riprap, longitudinal and transverse riprap dikes, bendway weirs, and willow posts.

Two general techniques are available for streambank erosion protection. One group of streambank erosion control practices involves the placement of structures or materials that resist the erosive force of flowing water directly against the bank. The second group involves an indirect approach using structures to divert or direct the erosive flow away from the bank or reduce the velocity of the flow immediately adjacent to the eroding streambank. The selection of the proper streambank erosion control practice depends upon the stream size, availability of materials, stream alignment, bank materials, stream substrate and sediment load, ecological factors, experience of the designer, and other factors.

Traditional bank protection materials include riprap, concrete paving, articulated concrete mattress, asphalt mix, vegetation, gabions, erosion control matting, and bulkheads (Thackston and Sneed 1982). More recently, recycled products and manufactured products including used automobile tires, concrete block products, plastic matting, and biodegradable matting are being used. Most of the products and materials require bank shaping to a uniform and stable configuration. Natural stone riprap can be used with or without bank shaping.

Longitudinal stone toe dikes placed at approximately 2 tons per linear foot along the toe of the bank have proven to be one of the most successful bank stabilization measures used in the DEC Program. Although these features are commonly referred to as a longitudinal dike, the primary purpose of the longitudinal stone toe is to resist the erosive force of the flow at the toe of the bank, similar to a revetment, and not to direct the flow as a traditional dike. Additional benefits of the longitudinal stone toe include the massive loading of the toe to resist rotational failure, and serving as a retaining wall for upslope failed material. The longitudinal stone toe is an excellent example of the bank toe control, or basal endpoint control, concept.

Transverse dikes generally fall under the category of a flow diversion structure. In the DEC Project, riprap flow diversion structures include traditional transverse dikes, shorter dikes referred to as hard points, and bendway weirs.

As originally conceived, bendway weirs are upstream-angled transverse dikes or sills that were constructed to be submerged, allowing navigation over the weir. The bendway weir concept was developed at WES on a moveable-bed physical model study for navigation on the Mississippi River for the U.S. Army Engineer District, St. Louis (Derrick et al. 1994). Bendway weirs were developed to widen the navigation channel in bends at locations where the natural point bar deposition encroached into the navigation channel.

Results of model tests indicated that bendway weirs widen a bend by increasing velocities on the point bar side and decreasing velocities on the outside of the bend. The resulting cross section approximates a rectangular section, not the expected triangular section of a natural bendway. Velocities are directed from the weir toward the point bar, and deposition of sediment occurs between the weirs on the outside of the bend, resulting in the flatter and wider section. Testing also indicated that the downstream crossing depth may be increased, perhaps from stabilization of the outside of the bend and storage of sediment in the flatter section.

As part of the DEC Project, the bendway weir concept was modified for application to small streambank protection. In a small stream, the weir would be submerged only during high flows. Although not a model of a specific bend, a sand bed and bank model test was conducted (Pokrefke 1993). The model stream had a width-to-depth ratio of 10 and a radius of curvature of 2.5 times the top bank width. The model reach consisted of six weirs, with the first weir located one bank top width downstream from the beginning of the bend. The longitudinal spacing of the weirs was 0.5 of the top width, with a weir length of 0.5 of the top width for one test, and 0.4 of the top width for the second test. Therefore, the weir spacing was one weir length apart for the first test, and 1.25 the weir length for the second test.

In the first test, the weirs were sloped from the riverward extent at the bed elevation to 2 ft above the bed at the dike root. In the second test, the riverward extent of the weir was 2 ft above the bed and sloped up to 4 ft above the bed at the root. Weirs were keyed into the bank in the manner used by the Vicksburg District and developed during the Section 32 Program and were angled 30 degrees upstream.

In the two tests, the weirs exhibited equal performance, and were effective in realigning flow and moving higher velocities away from the bank line and to the center of the channel. The longer weir, test 1, produced the greatest end scour and provided the greater length of downstream bank protection. The

shorter and higher weir, test 2, produced greater bank toe deposition.[1] No testing was conducted for comparison with traditional perpendicular dikes or with longitudinal stone toe.

Goodwin Creek

The Goodwin Creek study site, in Panola County, Mississippi, extends downstream from the Hubbard Road bridge at ARS measuring flume 2 for about 8,000 ft. Goodwin Creek watershed has an area of approximately 8.3 square miles, and has been studied extensively by the ARS Sedimentation Laboratory. In general, the stream is typical of the Yazoo Basin DEC streams; however, the Pleistocene mantle of loess overlies Citronella deposits of gravel and sand. Goodwin Creek, therefore, is a mixed-bed stream with significant portions of the bed material composed of gravel.

In a study conducted on Goodwin Creek from 1977 through 1983, erosion was documented using comparative cross-section surveys. The findings for the 6-year period in a stream reach approximately 12,700 ft long was that the net change in channel volume was 69,268 cu yd, or approximately 0.9 cu yd/linear foot/year. Analysis of the survey data indicated that 57 percent of the total bank erosion came from large bendways and 42 percent was produced by relatively straight, narrow channels. Typically, bank erosion in the narrow reaches was concentrated at sites where alternate bars or slight channel curvatures forced the flow to impinge directly onto the bank toe. The study concluded that in the long term, the randomly spaced, local erosion sites caused by slight irregularities will eventually stress the entire bank line. This implies that in the overall mission of the DEC, protection of straight channel reaches could be as vital as protecting the large bends, although less costly methods for the straight reaches should be investigated.

During the field inspection of Goodwin Creek, it was noted that riprap had been displaced from upstream bank stabilization and the upstream structure. Also, the baffle plate that had originally been constructed downstream of the structure has been removed. Without a baffle or energy dissipation basin, insufficient energy dissipation is occurring at the upstream drop structure, and the displaced riprap has been moved approximately 150 ft downstream. There was evidence that the riprap was also launched from the toe of the bank and was transported downstream.

The use of intermittent stone dikes in the relative sharp downstream bend has been successful. Much of the successful stabilization, however, is obstructed from view by kudzu. Scalloping between hardpoints was minimal. The longitudinal peaked stone toe protection constructed at the site is also covered with kudzu. The longitudinal peaked stone toe protection was

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[1] J. M. Carroll. (1995). "Hydraulic design of bendway weirs," Unpublished report, Colorado State University, Fort Collins, CO.

functioning without disturbance from the stream, and apparently no channel incision had occurred since these features were constructed.

In hindsight and with the amount of vegetation present, it is possible that a less costly type of stabilization could have been used. However, during original construction the conditions may have been much more severe than presently exist.

Little Bogue Creek

Little Bogue Creek is a tributary to Batupan Bogue, which is a tributary to the Yalobusha River. The study reach downstream extent is at a county road bridge crossing Little Bogue in Section 29, R7E, T21N, in Montgomery County. The study reach extends upstream through sections 28 and 27 approximately from station 500+00 to 660+00, which includes the confluence with Caffe Branch.

Bank stabilization was installed in 1987 by the Vicksburg District and SCS, and in 1991 by the Vicksburg District. An ARS-type low-drop structure was constructed in 1987 at approximate station 625+00 by the Vicksburg District, with weir crest el 267.0. Immediately downstream of the grade control structure, the SCS channelized a sinuous stream reach. A prominent outcrop of shale exists in the channel bed at approximately station 570+00. This outcrop has been, and continues to be, an important grade control in the channel.

Bank stabilization measures included transverse stone dikes; longitudinal peaked stone dikes of 1 and 2 tons per linear foot, with stone tiebacks; and full bank riprap revetment. Most of the stream length within the study reach has at least one bank with one of the listed riprap bank stabilization treatments.

The thalweg profile (Figure 59) indicates that the bed has degraded between stations 570+00 and 625+00 during the period between 1985 and 1991. This may have been influenced by construction of the upstream grade control. The 1995 data indicate that changes between 1991 and 1995 were minor in the reach immediately downstream of the structure. Upstream of the structure some filling is noted; however, the bed remains below the original weir elevation for a distance of approximately 3,000 ft. Recent reconstruction of the structure should improve upstream aggradation, and may have filled some of the deeper features immediately downstream of the structure. The slopes for the three channel reaches are listed in the accompanying tabulation.

Little Bogue	Thalweg Slopes
Lower reach	0.0012
Middle reach	0.0008
Upper reach	0.0017

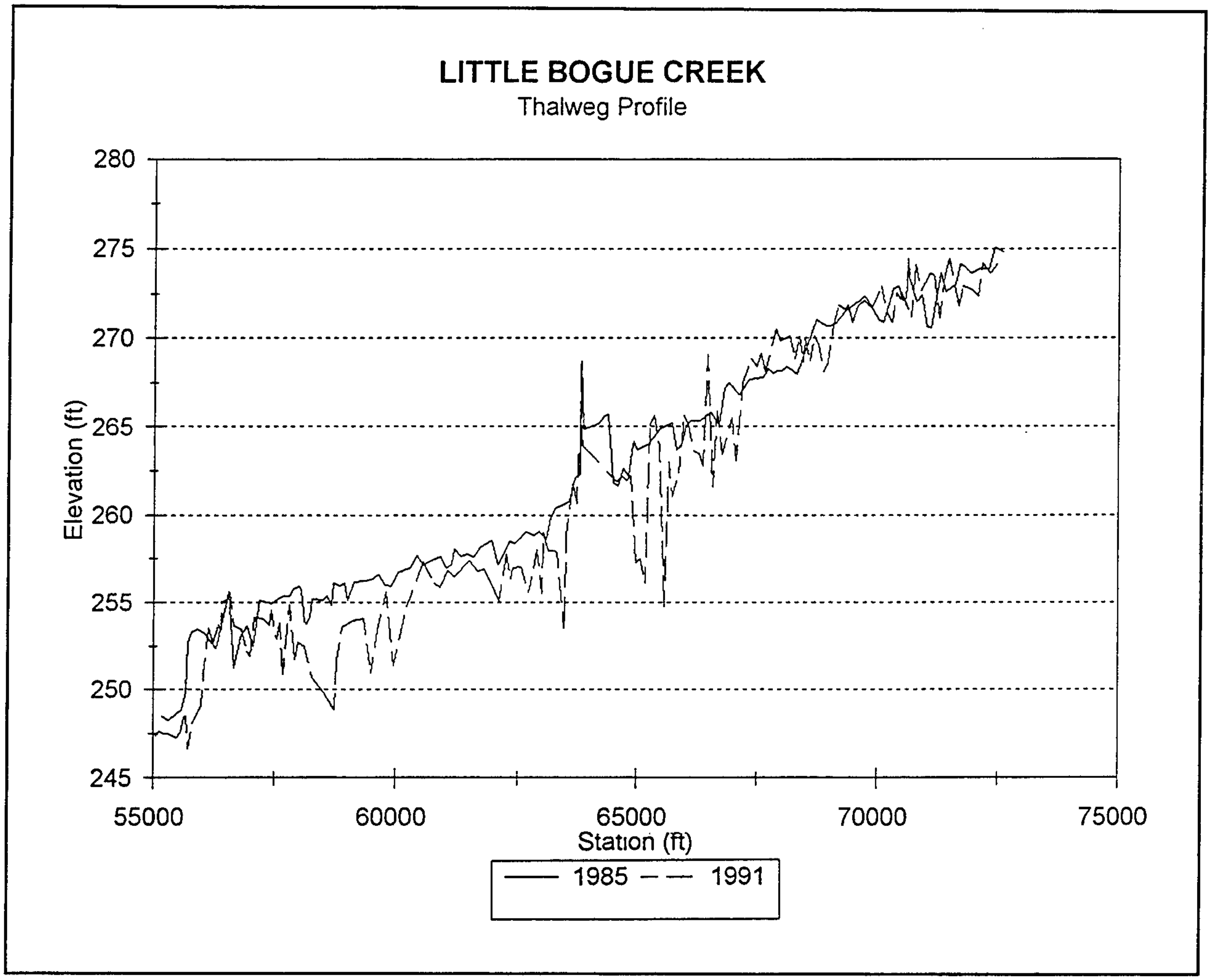

Figure 59. Comparison of 1985 and 1991 thalweg profiles

The channel bed is primarily sand contributed from eroding side channels and gullies. Although fluvial deposits were observed upstream of the grade control structure, the lack of filling upstream of the structure and the numerous reservoirs in the upstream watershed suggest that the upstream sediment yield has been controlled. Immediately downstream of the grade control structure on the left bank, a significant field drain and gully are supplying a significant sand source.

The field reconnaissance of Little Bogue began at the county road bridge near Wray's Store, at the downstream extent of the study reach. At several locations along the lower reach, small gullies provide significant sand deposits. The toe riprap in the lower reach is very unstable for walking, and frequently launches due to this disturbance. At several locations the streambed has been covered with riprap, either as a result of bank riprap launching, a construction access road, or as an intended grade control. Within this reach the right bank is protected by a stone toe and the left bank by a hardpoint, resulting in a scour hole and a very irregular bank downstream. The channel takes a more stable

appearance upstream, with some berming along the toe of the bank and with more permanent vegetation. In this reach the riprap is resting on the upper surface of the shale outcrop, and the shale has been eroded approximately 3 ft vertically. Launched riprap does not remain on the shale surface and is transported downstream. As shown in the thalweg profile (Figure 59), the water depth upstream of the outcrop is relatively deep.

Bank stabilization methods in the middle reach varied little from the methods used in the downstream, lower reach. Slopes are less, and the bed has been stabilized for many years by the resistant outcrop. The Little Bogue grade control structure had been damaged in previous years. During the period of this investigation, the structure was rehabilitated. Upstream of the structure, evidence of point bar development or other depositional forms indicating sand transport are difficult to discern through the vegetation. Wading the stream within the channel does reveal evidence of transport, and sediment supply appears limited.

In summary, the dominant feature of stream stability downstream of the structure is the shale outcrop. Little is known concerning the thickness or durability of the shale. Loss of the vertical control afforded by the shale would result in destabilization of the channel. Rock size for the lower reach appears to be too small, based on the amount of launching and transport noted; however, the rock may have provided a stable substrate long enough to allow vegetation to become the dominant bank stabilization. The middle reach will remain stable unless the shale outcrop fails, which would undermine the existing bank stabilization and threaten the grade control structure. The upper reach remains relatively stable, although cutoffs of the tight meanders may continue. Raising the weir crest and minor narrowing by the rehabilitation work may improve deposition upstream; however, it is expected to change only marginally.

Otoucalofa Creek

Otoucalofa Creek is a tributary to the Yocona River. Otoucalofa Creek has been channelized for the first 10 miles upstream of the confluence. The reach of interest is approximately 14 miles upstream of the confluence where Mt. Liberty Church road crosses the creek. The study reach extends 2,000 ft upstream and downstream of the Mt. Liberty Church road bridge. Two channel stabilization projects have recently been constructed within the reach, one in 1985 and a second in 1989. Additionally, toe riprap and spur dikes were observed in the field that were not part of either project. It is assumed that these were placed prior to 1985.

Otoucalofa Creek has a width of approximately 100 to 150 ft and a depth of 10 to 15 ft. Approximately 3 miles downstream of the study reach, the creek has been channelized for 10 miles to the confluence with the Yocona River. The channel is actively incising within the study reach probably as a result of the channelization downstream. The significant sand and gravel transport

through the reach is estimated to be approximately 21,500 tons per day at the 2-year discharge. This equates to a sediment concentration of approximately 1,725 mg/ℓ. The 2-year discharge is estimated to be 4,600 cfs. The average sediment size (D_{50}) is 0.4 mm, including fine gravel up to approximately 10 mm. The bank material comprises primarily sand with sufficient amounts of silt and clay material such that steep banks remain stable.

In 1985, six spur dikes were constructed along the left bank of the channel downstream of the bridge, and five spur dikes were constructed upstream of the bridge. The dikes are spaced approximately 100 ft on center. On the right bank of the channel toe, riprap was placed for the first 500 ft upstream of the bridge at the rate of 1 ton per linear foot. It appears that the riprap placed in 1985 was to prevent lateral movement of the channel and protect the bridge. In 1989, additional channel stabilization measures were designed upstream of the bridge to prevent further lateral channel migration. The channel stabilization measures start approximately 500 ft upstream of the bridge where the 1985 construction ended. The 1989 design extends to 3,500 ft upstream of the bridge. A series of 17 spur dikes were constructed along the left bank and an equal number of dikes were constructed on the right bank. In addition, toe riprap was used to help stabilize the banks. On the left bank, 800 ft of riprap were placed at the rate of 1 ton per linear foot, and on the right bank, 700 ft of riprap were placed at the rate of 1 ton per linear foot. No measures were taken to stabilize the bed of the stream at that time.

The two downstream left bank dikes are experiencing severe scalloping; however, they are remaining functional. There does not appear to have been much lateral movement of the channel. The toe riprap and dikes are stable and have not been displaced by the streamflow. This indicates that the rock was properly sized and constructed. Sediment has deposited behind much of the toe riprap and between some of the dikes.

The two channel stabilization projects provide lateral control of the channel; however, no grade control for the channel was included. Consequently, the bed elevation of the channel is continuing to decrease as the channel further incises. One to two feet of bed degradation have been observed in the 2,000-ft reach downstream of the bridge over the past 3 years. This is evidenced by thalweg surveys conducted in 1992, 1993, 1994, and 1995. The problems resulting from the continued degradation are becoming evident in the field. In many locations, the toe riprap placed along the banks of the channel is elevated above the present channel invert. A vertical drop of approximately 1 ft from the bottom of the toe riprap to the streambed was observed in many locations. At some locations, the riprap is launching into the stream. The spur dikes are also vulnerable to continued incision of the channel with the low-flow channel incised below the bottom of the dike. As the incision continues, the low-flow channel will widen and undermine the dike. The result will be a launching of the stone, and ultimately, a complete failure of the dike.

In the upstream 1,500 ft of the channel, the streambed is composed of a hard clay material. The continuing incision of the channel has been retarded

for a period while the clay is being eroded. There is presently a head cut in the hard clay approximately 2,000 ft upstream of the bridge. The drop across the entire head cut is approximately 1 to 2 ft in a 100-ft reach of channel. However, it is estimated that the clay lens is only 1 to 2 ft thick. Once the clay has been eroded, a more rapid rate of channel degradation is expected. Figure 60 shows a thalweg profile of the creek based on the surveys conducted in 1992, 1993, 1994, and 1995. A trend can be seen that shows a continued lowering of the bed elevation.

In summary, there does not appear to have been any substantial lateral movement of the channel within the study reach since the two channel stabilization projects were constructed. The riprap is typically remaining in place and is not being removed by high flows in the channel. However, the two channel stabilization projects do not appear to have had any impact on the rate of incision of the channel, and the channel is continuing to degrade. As a result, in some locations, the toe riprap is starting to launch and the spur dikes are being undermined. If the stream continues to incise, the integrity and stability of the toe riprap and dikes will be compromised. It is recommended that action be taken to stop further incision of Otoucalofa Creek.

Red Banks Creek

Red Banks Creek has a width of approximately 150 to 200 ft and an average depth of 15 ft. The channel is relatively straight and has generally uniform characteristics along the 13,100-ft study reach. This is the result of channelization of the creek. The significant sand and gravel transport through the reach is estimated to be approximately 14,000 tons per day at the 2-year discharge. The 2-year discharge is estimated to be 4,000 cfs. The average sediment size (D_{50}) is 0.5 mm, including fine gravel up to approximately 4 mm.

Red Banks Creek is an oversteepened channel that is actively incising. The oversteepening is the result of channelization. The resulting incision is approximately 15 ft deep and is increasing. At present, it is estimated that the channel capacity exceeds the 100-year event, which is estimated at approximately 12,000 cfs. The bank material is composed primarily of sand with a sufficient amount of silt and clay that the very steep banks remain relatively stable. The soil characteristics in the basin can be characterized as group B and group C using the SCS soil classification method. The scale ranges from A to D, with group A soils having the lowest runoff potential and group D soils having the highest runoff potential.

The study reach extends approximately 12,000 ft upstream from the Highway 309 bridge. In 1991, 4,000 ft of toe riprap was placed along the right bank. The riprap was placed at the rate of 1 ton per linear foot in lengths ranging from 150 to 600 ft. A total of 12 segments of toe riprap were placed along the right bank of the channel. The toe riprap was tied into the banks of the channel through a series of 28 tiebacks. The tiebacks are located at the beginning and end of each riprap segment. Long segments of riprap may also

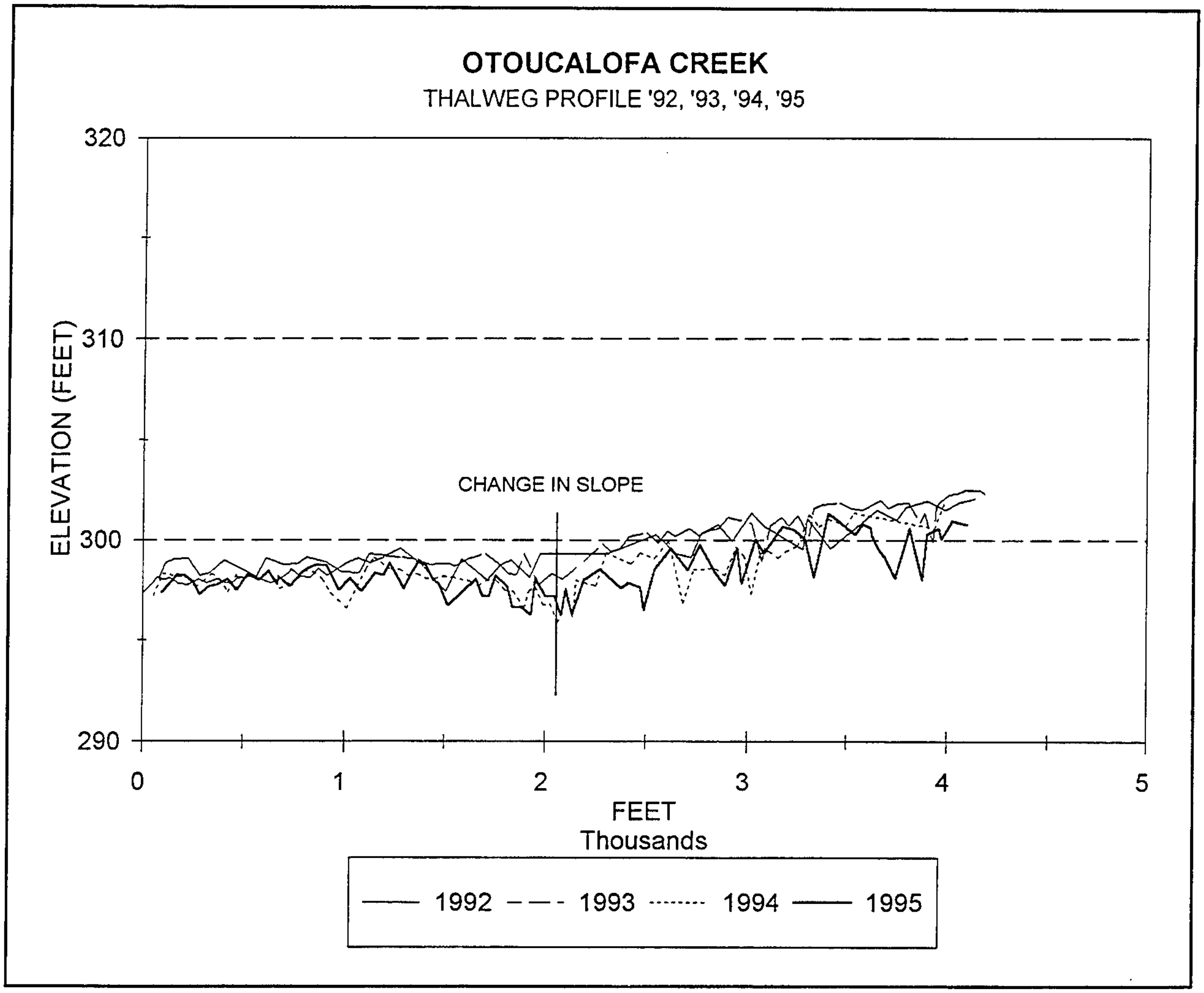

Figure 60. Thalweg profiles, 1992-1995

have tiebacks between the ends. A total of 7,800 lin ft of toe riprap were placed along the left bank of the channel. The riprap was placed at the rate of 1 ton per linear foot in lengths ranging from 400 to 2,200 ft. A total of eight segments of riprap were placed along the left bank of the channel. The riprap was tied to the banks with a series of 28 tiebacks.

A series of five v-notch weirs were constructed in the 12,000-ft study reach. The weirs were constructed of riprap placed across the channel. Filter cloth was placed between the riprap and the underlying material. The weirs are located 4,000, 5,500, 7,000, 8,375, and 10,000 ft upstream of the bridge. Each weir provides a 2-ft rise on the bed of the channel. The typical weir length is 14 ft. The weirs are constructed of R-1000 stone and designed to serve as grade control structures. One hundred feet of toe riprap were placed along each bank upstream and downstream of each weir. Stone tiebacks were used at each end of the toe riprap and at the weir.

For approximately the first 1,000 ft upstream of the bridge, the toe riprap was placed along the right bank approximately 20 ft from the toe of the slope. Five tiebacks were used along the right bank to tie the riprap to the banks. Toe riprap was also placed along the left bank for approximately the first 2,000 ft upstream of the bridge. In the first 1,000 ft, five tiebacks were constructed on the left bank.

The design intent is that sediment would deposit in the slack water between the longitudinal riprap and the toe of the slope. The deposition of sediment would effectively reduce the height of the banks and help to stabilize them. As a result of the increased bank stability, it would be possible for vegetation to colonize on the banks. The vegetation would further reduce the water velocities over the banks, and slower growing willows and ultimately trees would grow on the banks of the channel.

One consideration when using this technique is that the vegetation will effectively decrease the channel width. The width adjustment will be accompanied by a slope adjustment such that the modified reach will transport the same amount of sediment as is supplied to it from upstream. A HEC-6 model of the stream can be used to help predict long-term channel response to the change in bank roughness. If a decrease in bed elevation is anticipated, grade control structures should be built. A substantial decrease in bed elevation could undermine the toe riprap, causing it to launch and possibly destabilize the banks.

The impacts of the toe riprap and dikes for the first 1,000 ft upstream of the bridge have been as anticipated. Sediment has deposited between the riprap and the banks, and vegetation has taken hold. Willow trees are starting to grow on the right bank and sediment is continuing to deposit between the toe riprap and the toe of the slope. Based on thalweg surveys conducted in 1993, 1994, and 1995, there does not appear to be any conclusive evidence that the channel has started to scour as a result of the increasing bank roughness (Figure 61).

The increase in bank roughness has very little impact on the sediment transport capacity of the reach during periods of low flow. Only during periods of high flow, greater than 1,000 cfs, will the depth of flow in the channel be sufficient that the bank vegetation has a major impact on the overall channel roughness. Red Banks Creek is very similar to other small streams in the Yazoo Basin and has a very flashy hydrography. Base flows are estimated at 100 to 200 cfs. The 2-year event is estimated to be 4,000 cfs. At a discharge of 4,000 cfs, the depth of flow in the first 1,000 ft upstream of the bridge is approximately 6 ft. This indicates that a flow on the order of the 2-year event or larger is required before a significant change in the sediment transport capacity of the cross section can be anticipated.

A HEC-6 analysis was used to predict the change in channel aggradation or degradation resulting from the increase in bank roughness. The length of the surveyed reach was approximately 13,000 ft. Cross sections were surveyed

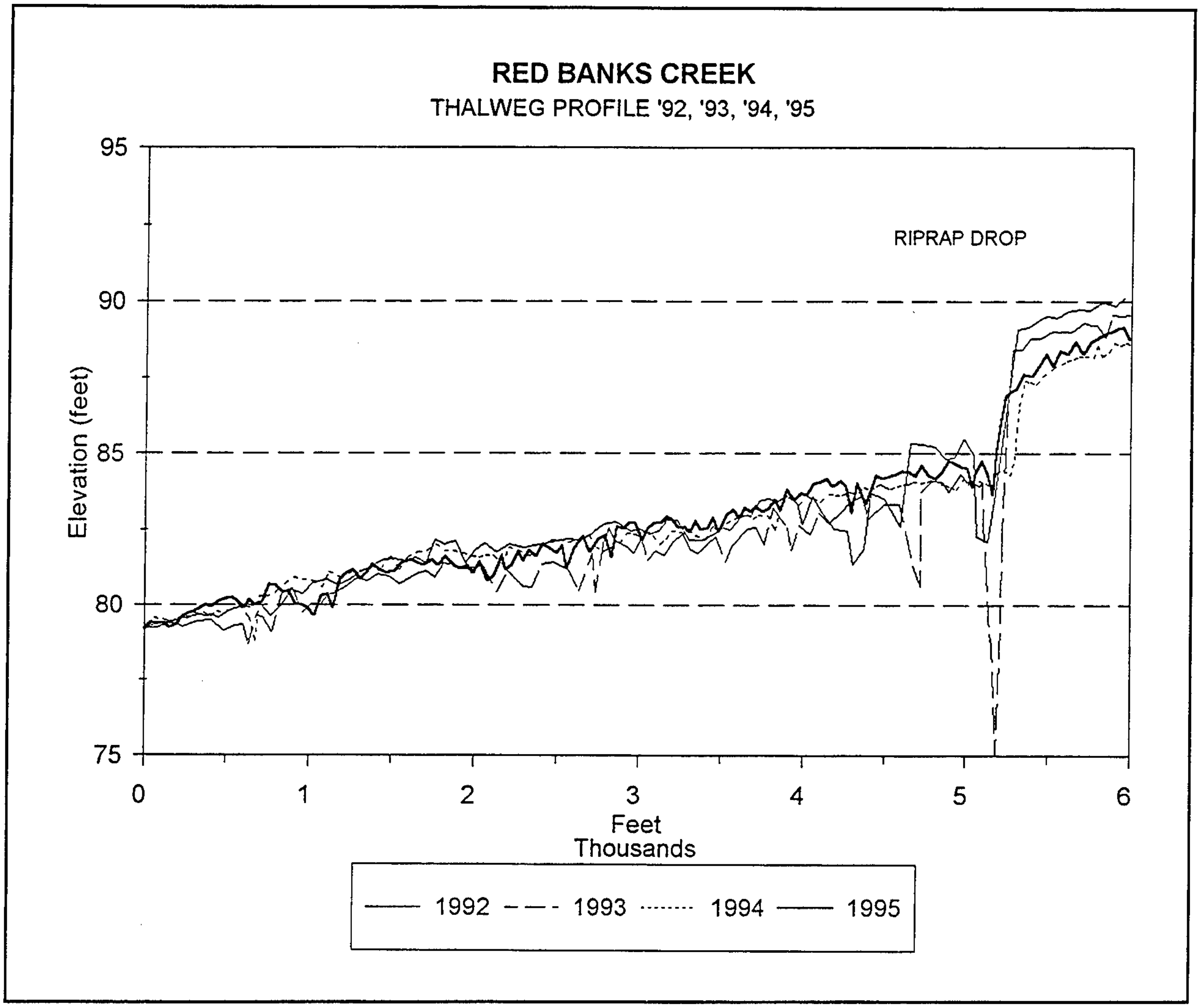

Figure 61. Thalweg profiles surveyed in 1992-1995

every 500 ft. The change in bank roughness occurred only in the furthest downstream 1,000 ft of the channel. In the downstream 1,000 ft of the channel, interpolated cross sections were added to the surveyed cross sections. The cross-section spacing was 100 ft in this channel portion. The initial depth of flow at the downstream end of the channel was assumed to be normal.

Two constant discharge rates were used in the HEC-6 analysis, 750 cfs and 4,000 cfs. The lower discharge is the amount of flow required to fill the channel to the top of the toe riprap, and the higher discharge is the estimated 2-year event. At a discharge of 750 cfs, the simulation was run for 1, 7, 14, and 28 days. At a discharge of 4,000 cfs, the simulation was run for 1, 7, and 14 days. All seven simulations were run for the low bank roughness and the high bank roughness for a total of 14 simulations. The bank roughness values were estimated from a comparison of field slides with values in Chow (1959). The sediment inflow into the most upstream cross section must be estimated for the simulation. The simulation should be insensitive to the sediment inflow

since the reach of interest is 12,000 ft downstream of the upstream end. However, to ensure that this was the case, all 14 simulations were repeated with a sediment inflow that was half of the estimated sediment inflow. It was found that the simulations were insensitive to the estimated inflowing sediment transport rate.

The results of the simulation showed that the increase in bank roughness has a minimal impact on the predicted channel aggradation or degradation. Figure 62 shows the channel profile after 7 days of flow at 4,000 cfs. The bed profile prior to the simulation is shown along with the bed profile for the low bank roughness and the high bank roughness after the simulation. Only the downstream 6,000 ft of the channel are shown in Figure 62. The simulation shows approximately 1 to 2 ft of bed degradation between stations 5+00 and 20+00. This is due to the fact that the channel in general is narrower in this reach than it is between stations 0+00 and 5+00. The reduced channel width occurs naturally and is not the result of any channel stabilization measures. The model showed slight bed aggradation between stations 0+00 and 5+00 for the low bank roughness and no real change for the high bank roughness. The difference between the low and high bank roughness, however, is less than 1 ft. Considering that the 2-year event probably has a duration of less than 2 days, the difference can be considered negligible.

Figure 61 shows the thalweg profiles as surveyed in 1992, 1993, 1994, and 1995. The profile shows the channel reach from station 0+00 to 60+00, the same reach shown in Figure 62. Based on the thalweg surveys, there does not appear to have been any significant change in the thalweg profile over the past 4 years.

In summary, on Red Banks Creek, toe riprap and spur dikes have been used successfully to stabilize the banks of the channel. The increase in bank roughness does not appear as though it will significantly changed the future aggradation or degradation of the channel in comparison to degradation that would have occurred without the riprap. This is at least in part due to the fact that the channel is much deeper than that required to contain the 2-year flow. The increase in bank stability due to the growth of vegetation will result in a reduction in the sediment supply.

Harland Creek

Harland Creek is tributary to Black Creek. The average width of Harland Creek is approximately 95 ft and average depth is approximately 6 ft; however, these dimensions vary significantly along the reach. The 2-year discharge is approximately 3,750 cfs. The channel has an actively meandering, sinuous planform, as shown in Figure 63. The significant sand and gravel transport through the reach is estimated to be approximately 8,700 tons per day at the 2-year discharge. The average sediment size (D_{50}) is 0.5 mm, including gravel up to approximately 4 cm.

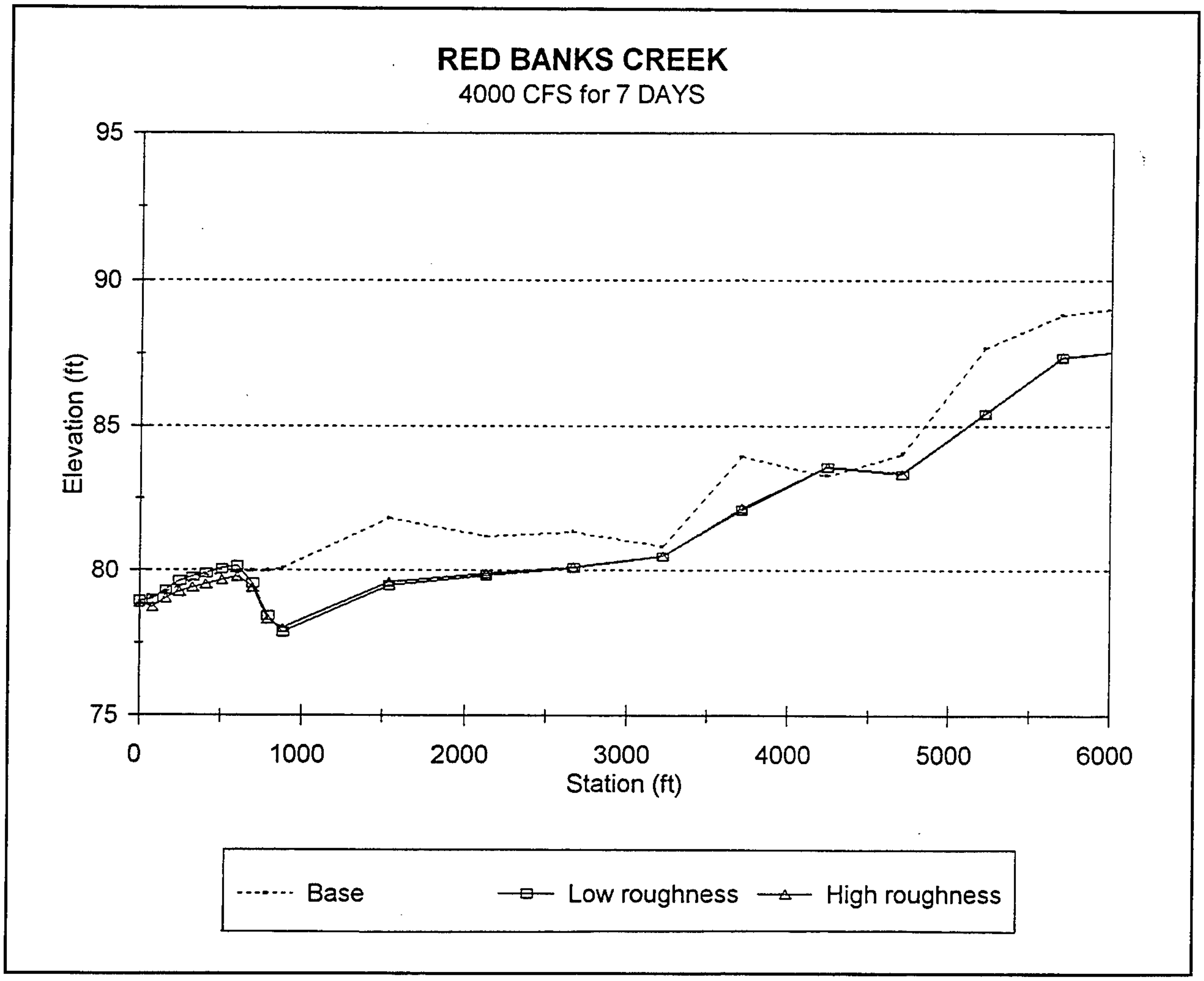

Figure 62. Comparison of predicted thalweg profiles with low and high bank roughness

Harland Creek has been an extremely active meandering channel as can be observed from the bank line comparison (Figure 64) made from four sets of aerial photographs dated 1955 to 1991. Based on measurements from the aerial photographs, the average annual bank migration rate is approximately 14 ft per year, or about 15 percent of the bendway channel width annually. To the landowner, this translates to movement of a full channel width in less than 7 years, approximately 12 acres of land loss per mile of stream.

Figure 65 is a plot of Harland Creek data showing the radius of curvature of the meander bends divided by the channel width on the x-axis, and the annual migration rate divided by the channel width on the y-axis. The dark, filled data points are representative of the upper envelope of data points for the Beatton River, British Columbia (Hickin and Nanson 1975). The envelope lines indicate that the maximum rate of channel bend migration occurs as the ratio of the radius of curvature to width (R_c/W) approaches a value of 3.0, or in a

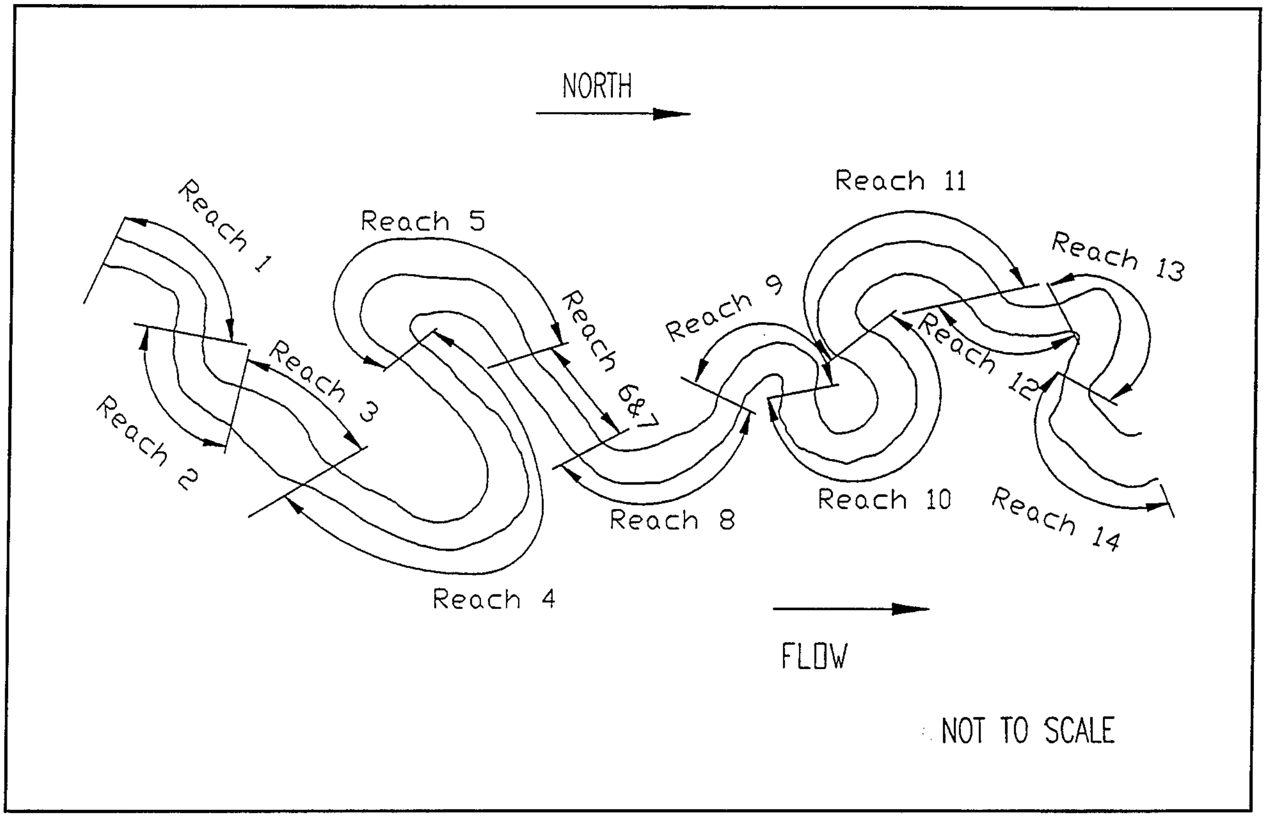

Figure 63. Harland Creek general planform

broader range, for about 2.5 to 3.2. Bends in this range are rapidly altered to either a greater or less R_c/W, which have less dynamic rates of migration. Less than 20 percent of the Harland Creek data were found in the 2.5 to 3.2 range.

Another striking similarity between the Beatton River and Harland Creek data sets is the average value of the R_c/W value: 2.17 for Harland Creek and 2.11 for the Beatton River. Leopold and Wolman (1957) developed two morphometric relationships for channel wavelength that gave an average value of R_c/W of 2.3. Although the Beatton River, Harland Creek, and the Leopold and Wolman data are from diverse rivers, these similarities exist, suggesting some controlling hydraulic relationship.

In July and November 1994, site visits were made to Harland Creek. Significant flooding during the investigation has occurred, providing a basis for assessment. An initial observation between July and November was the lack of growth and survival of the willow posts. Additional detailed counts were made to establish willow post mortality.

Stone displacement due to fluvial transport in the downstream willow post/bendway weir site and at the upstream longitudinal stone toe site was observed to be minimal. Some rock movement occurred at scalloping locations

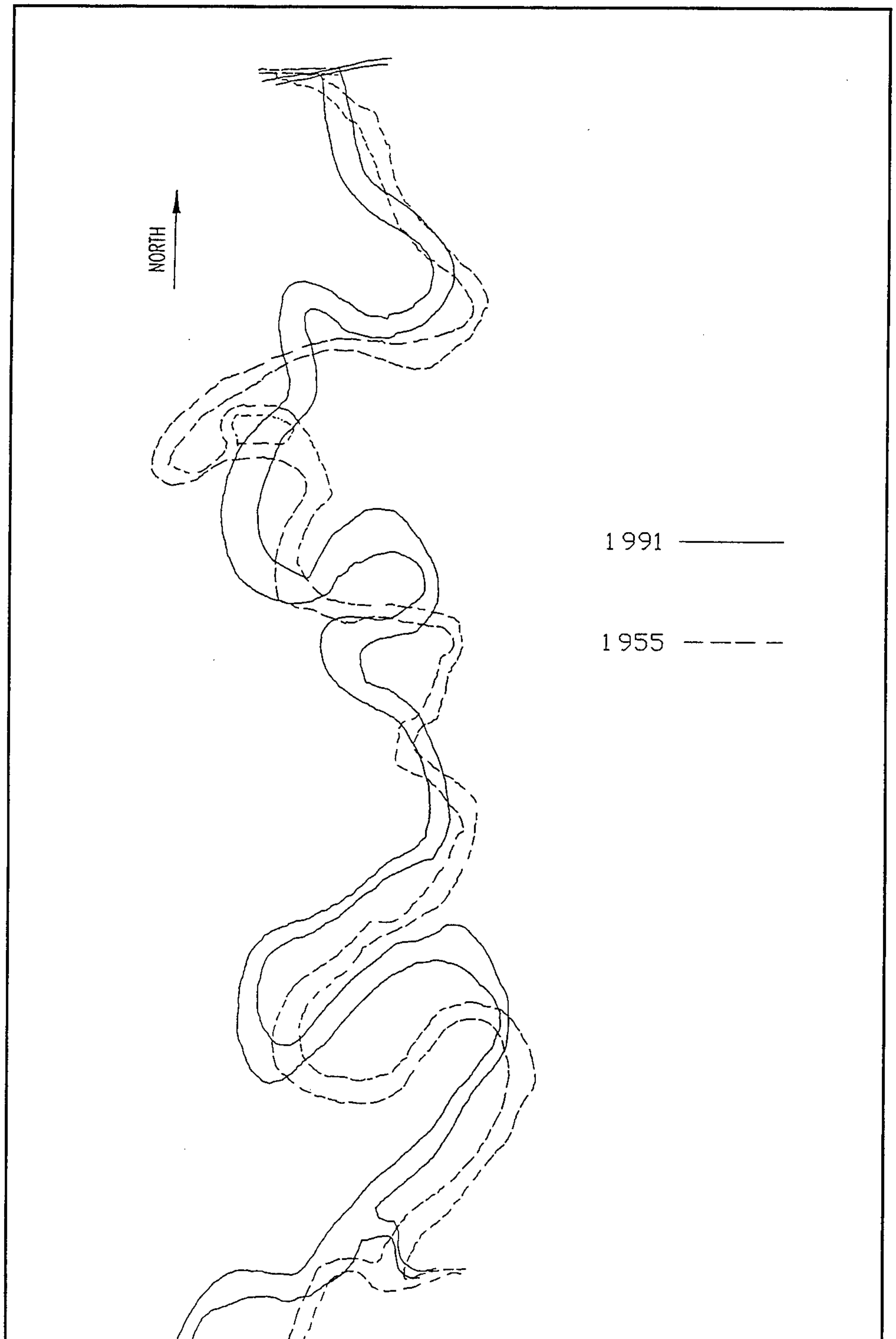

Figure 64. Harland Creek, bank line comparison from 1955 to 1991

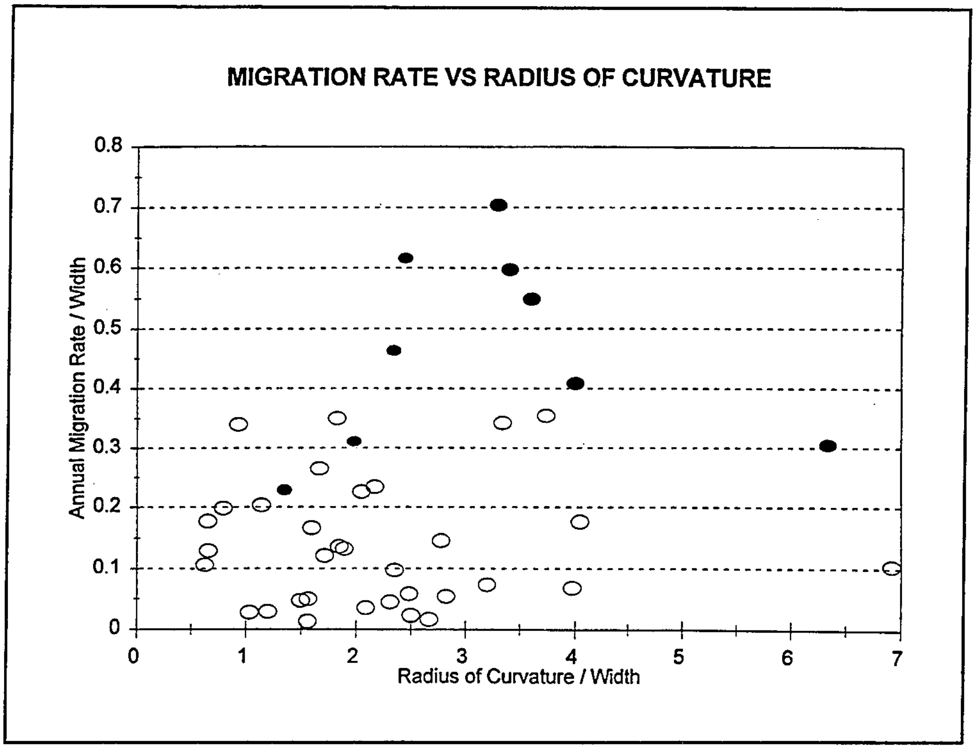

Figure 65. Harland Creek, bendway radius of curvature versus migration rate

between the bendway weirs, and at transitions between sections of the longitudinal stone toe at the upstream site. The upstream Harland Creek site experienced insignificant movement and bank line change in the protected banks compared with the downstream willow post/bendway weir site. However, the unprotected portion of the upstream site was severely eroded during the period. It must be noted that the downstream willow post/bendway weir site relied upon two experimental bank stabilization techniques. The site was the first application of either technique by the Vicksburg District in small, incised meandering channels.

Field observations indicated that the upstream and downstream weirs in each system were approximately perpendicular to the flow. The angle of each adjacent structure increased toward the midpoint of the bend, to no more than 20 degrees upstream of perpendicular to the flow. The angle and spacing of some weirs observed in the field varied from the design, a result of problems during construction layout. These observations point out the additional complexity in designing and constructing a dike or bendway weir bank stabilization system, compared with a longitudinal toe and willow post system. Dike length, spacing, and bendway weir angle with the flow are critical factors that contribute to the success or failure of the system. In Reach 1 (Figure 63) incipient flanking of the downstream weir was observed, and a portion of the tieback had launched. In several of the weir systems, the downstream weir was severely stressed. Several weirs in Reach 2 were observed to be approximately perpendicular to the flow, and scalloping between these weirs was greater than

between the upstream angled weirs. However, low-level bars were observed to be forming and connecting between adjacent weirs. In the transition between Reach 2 and Reach 3, bendway weirs on opposite banks actually overlapped, resulting in a zero projected channel width. A large gravel midbar had formed at this location in July, and by November had attached to the right bank. A single bendway weir is located in the Reach 4 bend, and is forcing the chute channel to develop away from the outer channel. In Reach 4, the riverward row of willows, which generally do not survive and are considered sacrificial, is functioning to protect the upper rows from debris and to protect the toe. In Reach 5, where a combination of 1 ton per linear foot of riprap and willow posts to stabilize the toe of a loess bluff was used, bank stabilization has been acceptable. Reaches 6 and 7 had good survival of willow posts and were relatively stable. Reach 8 has had significant toe scour in the willow posts, which eroded the soil away from the first row of willow posts and resulted in high mortality. Reaches 9 and 10 are in an incipient neck cutoff. The initial bendway weir system at this site required reinforcing using a 2-ton- per-linear-foot longitudinal toe. Experience has shown that the use of dikes in very tight radius bends is a high-risk application. Protection of the neck cutoff could be enhanced by planting thicket-type vegetation in the neck area, perhaps privet. Bendway weir construction in Reach 11 has been generally successful to date, except at the downstream extent where some repair was required. Relic tension cracks that formed prior to construction are now vegetated and remain visible. Reach 13 was protected using only willow posts. The willow posts have been undermined and are falling into the stream. This reach has now been reinforced using a riprap toe.

Figure 66 is a bar chart indicating the value of the radius of curvature to channel width ratio for each of the eleven bends within the fourteen stabilized reaches on Harland Creek. The labels on the bars indicate the type of stabilization that was emplaced. On bars 9 and 13, it is noted that the initial stabilization was repaired using a 2-ton-per-linear-foot longitudinal stone toe. The measured value of the R_c/W ratio was less than 0.65 for both of the bends requiring repair.

Reach 9 was initially stabilized using bendway weirs. Due to flanking of the weir and continued bank erosion at the downstream extent of the weir field, repair was made using the stone toe. Apparently, alignment and spacing of the bendway weirs, as constructed, were not adequate to control the erosion. It has been observed that traditional dikes require very close spacing on tight bends, and the necessity of repair in Reach 9 is an example of this malady for bendway weirs. No evidence was noted that the stone was too small for the imposed shear stress. In Reach 13 the four rows of willows failed due to excessive scour of the toe material. Willows were exposed to the flow with no soil covering the potential roots.

With the exception of Reach 9, bendway weirs have functioned adequately with R_c/W values of from 1.89 to 2.77. No repair of willow post stabilization has been required with R_c/W values from 1.03 to 1.50, except on Reach 13.

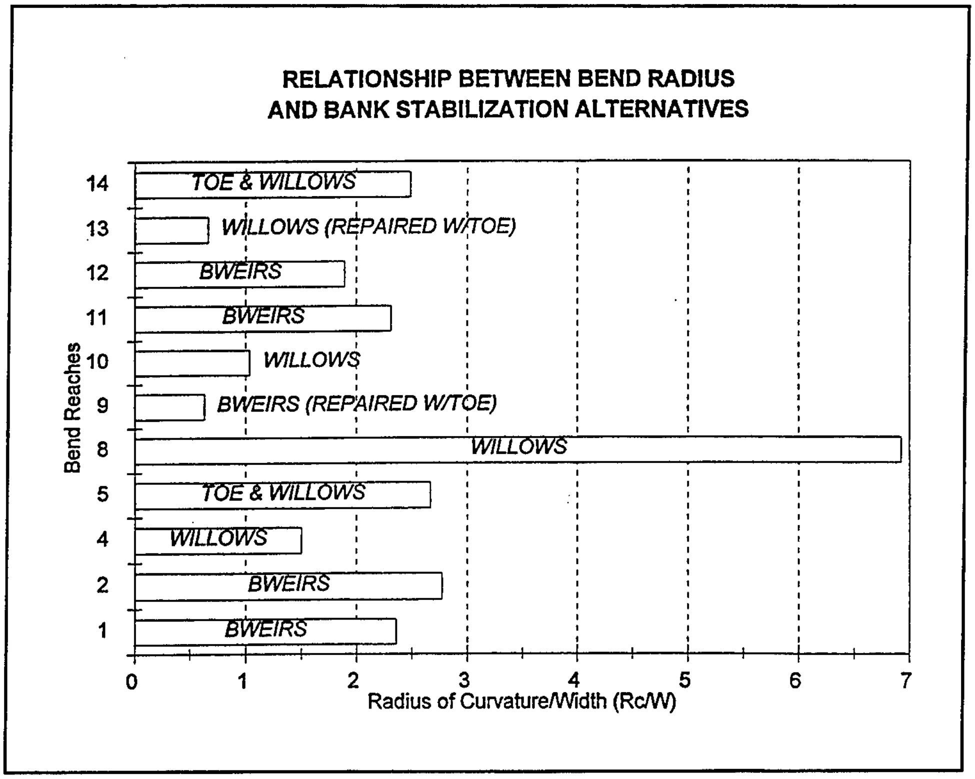

Figure 66. Relationship between R_c/W and bank stabilization alternatives

Although it is too early to be certain, it appears that the willow post stabilization should function in the average range of values of R_c/W, i.e., 2.0 to 2.3.

The willow posts were initially installed in February 1994. By June 1994 the survival rate of the willow posts was measured at 80 percent and by October 1994, 42 percent. Therefore, an evaluation of the Harland Creek sites to assess the survival rate of the willow posts was conducted. The data gathered consisted of the post diameter, elevation above or below the low-water level at the ground line of the willows, the aspect of the streambank, canopy above the stream, and a count of the willows that were dead and alive.

In 1994, the diameter of 293 dead and living willow posts was measured. The diameter of these posts ranged from 2 to 6.5 in. Each post was placed into a class based on diameter. The percent alive was then plotted versus the lower boundary of each class, as shown in Figure 67. The data indicate that the survivability increases as the diameter increases from 1 to 7 in.

Similar to the diameter, the elevation at the base of the post above low water was measured for each of the 293 posts in the study reach. These elevations ranged from 2 ft below the water surface to 7.5 ft above the water surface level. The data were divided into classes of 1-ft increments, as shown in Figure 68. The data indicate that survivability reaches a minimum at the lower and upper ranges of the data. At several locations it was evident that scour of

the toe of the bank resulted in exposing posts, thus reducing the elevation of the base of the post.

The aspect of the streambank, or the direction the bank faces, was measured using a Brunton compass. The aspect of the streambank data indicate that banks facing east or west have a greatest survival rate, and that banks facing north are better than banks facing south. Figure 69 portrays the aspect data.

Data for the effect of canopy were inconclusive indicating that the amount of canopy may not be a primary factor within the limits of the data measured for the Harland Creek site.

Survival of the willow post is obviously necessary for long-term viability of the installation. The high mortality of willow posts can be attributed to insect infestation, lack of aerobic rooting conditions caused by impermeable soils, damage from high velocity, sediment-laden flood flows abrading the new growth, improper backfilling during construction, lack of sufficient moisture, and constant anaerobic rooting conditions due to standing water. The Harland Creek data suggest that the high mortality of the lower row of posts was the result of streamflow damage and the anaerobic conditions in standing water or poor soils. The high mortality of the upper rows, more than 6 ft above low water, may represent a site-specific threshold of sufficient moisture for sustaining growth.

Sprouting of new willow shoots adjacent to dead posts was frequently observed. Other woody and nonwoody species were observed to be colonizing within the willow post sites. Because of the increased hydraulic roughness within the sites, sediment was observed to be depositing and was providing suitable conditions for riparian invasive species. In addition, many of the posts, alive or dead, contributed to the stability of the bank by increasing the bank strength and by holding failed soil blocks.

Relative to the use of willow posts on Harland Creek for bank stabilization, the following conclusions are presented:

a. The cost of willow post construction is an attractive advantage over riprap and many other bioengineered methods.

b. Dominant factors affecting the Harland Creek willow post installation include post base height above low water, post diameter, and aspect.

c. Scour at the toe of the willow post installation should be addressed at future sites.

d. Total survivability of 80 percent was measured in June 1994, and 42 percent was measured in October 1994.

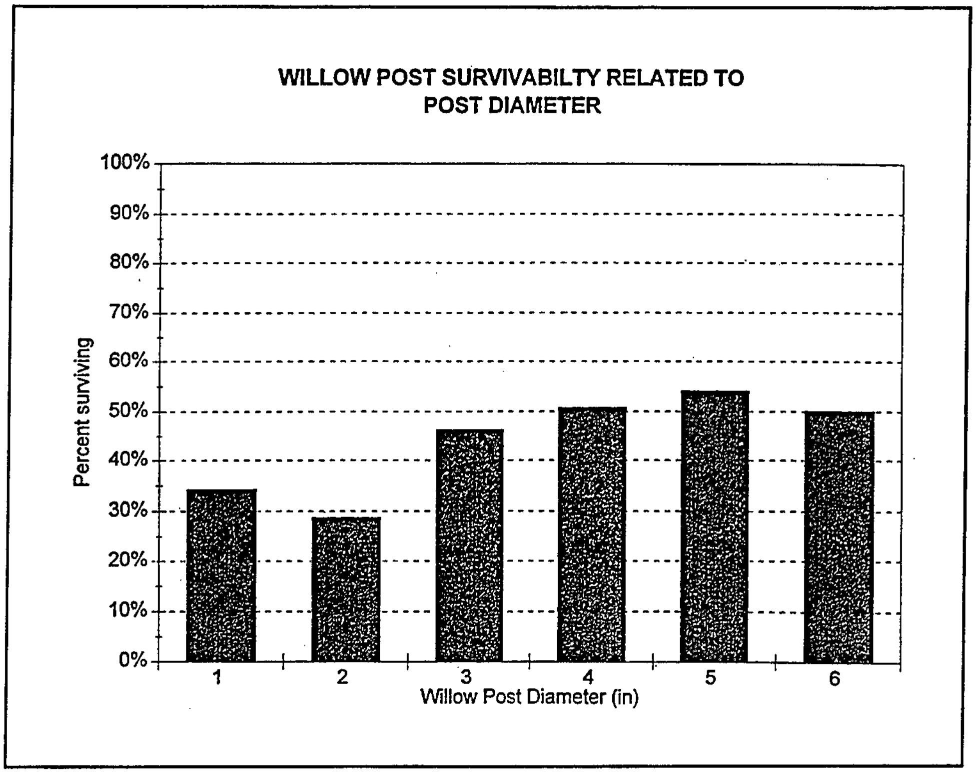

Figure 67. Willow post survivability related to post diameter

Along with the recommended installation guidelines (Raphelt et al. 1995), the survivability of the willow posts at the Harland Creek site could have been improved by preventing erosion of the toe of the bank during the establishment period. Natural stone riprap placed at 1 ton per linear foot in a peaked longitudinal stone toe, continuous toe reinforcement using anchored cedar or willow trees, or use of manufactured products such as heavy fiber rolls placed along the toe, could be of benefit to system survivability. Interplanting of the willow posts with other species and the use of fascines buried along the rows would be of benefit in enhancing aggradation within the post system and could provide rooting medium in areas of poor soil conditions.

In summary, continued development of the willow post technique is strongly recommended due to potential cost savings, relatively low disturbance to habitat, and low mobilization costs. The experimental application of willow posts and bendway weirs has shown great potential, particularly for cost reduction when combined with the longitudinal stone toe. Bendway weirs have been shown to be successful. However, the cost effectiveness of a combination of willow posts supplemented by 1 ton per linear foot of toe riprap in high-stress locations, together with the higher risk of dike design and construction should be strongly considered.

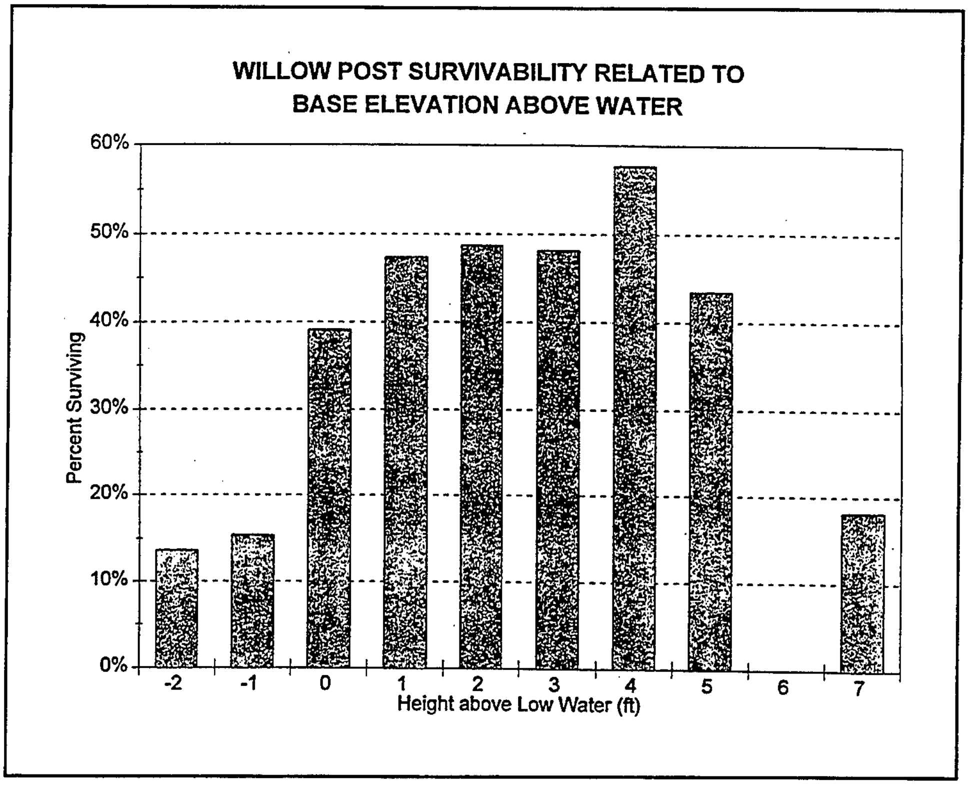

Figure 68. Willow post survivability as function of base elevation above low water

WES Evaluation of Harland Creek Bendway Weirs and Willow Post Test Site

Introduction

The previous evaluation on the bendway weirs and willow post techniques used on Harland Creek was made by CSU. This section will present the evaluation conducted by WES. The approach of conducting two evaluations was based on the desire to obtain independent and reasonable evaluation of the performance of the two techniques. Mr. David L. Derrick was the principal investigator for the WES evaluation and the principal designer for the two techniques used on Harland Creek. Thus this monitoring effort provided him the opportunity to see the successes and failures of the particular reaches that he had designed.

At the end of FY 1993 and beginning of FY 1994, portions of Harland Creek near Tchula, MS, were stabilized using a combination of bendway weirs and willow posts. The installed plan included installation of bendway weirs, willow posts, a combination of weirs and willow posts, or a combination of willow posts and longitudinal peaked stone dikes. The design and layout of those features are presented in Raphelt et al. (1995). The weirs were installed in the fall of 1993 and the willow posts near the beginning of 1994.

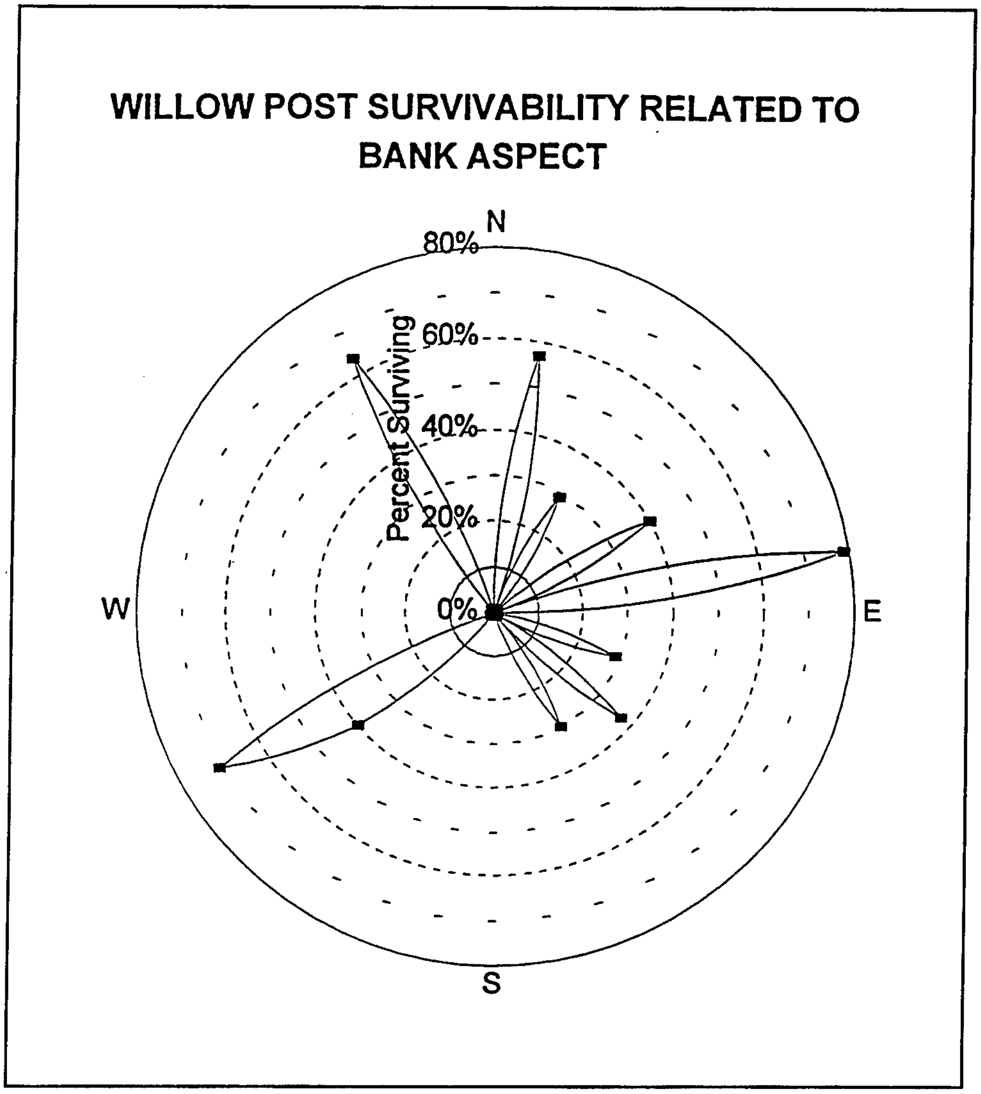

Figure 69. Willow post survivability as function of bank aspect

The Harland Creek reach used to evaluate the effectiveness of alternative bank stabilization methods is 11,700 ft long and encompasses 14 bends with steep (5 to 44 ft high) actively eroding banks. Additionally, the ARS has provided technical assistance in identifying the stream features needed for successful aquatic habitat improvement or restoration. Those features were included in the design of the various plans constructed. During the 1994 to 1996 period ARS will monitor the post-project stream for habitat improvement and fish species diversity, density, and size (Raphelt et al. 1995).

Bendway weirs

Of the 54 weirs constructed, 35 were initially located and angled incorrectly. Since construction, 5 of the weirs have been relocated, 11 weirs were reangled, and 19 weirs have not been modified and are still incorrectly

located or angled to some degree. Also, since the weirs were constructed, the site was subjected to at least four out-of-bank flows in the first year. In spite of these conditions and events, the weirs have generally performed very well in stabilizing the outer bank of the bends. Most banks are rapidly maturing as a mixture of plants (grasses and weeds) and volunteer willows have invaded these areas.

Excessive bank scour and scalloping have occurred between the last two weirs and downstream of the last weir in each bend with weirs. In the six reaches with weirs, the scour between the last two weirs has naturally healed and stabilized. The bank instability downstream of the last weir has naturally stabilized in four of the six bends. One of the two problem areas (Reach 9) was caused by contractor error where one weir was left out and the immediate upstream and downstream weirs were incorrectly angled between 35 and 45 degrees upstream. This situation has created a complex set of factors causing the problems experienced in Reach 9. Three main contributors are typically the following: a bend is under more pressure from the apex to the downstream end; stone from the weir key protruding from the surrounding bank appears to cause some local turbulence and scalloping; and uncontrolled high flows across the point bar impact the bank and incorrectly enter the weir field. That particular flow condition causes the flow to be angled into the weirs where the weirs cannot redirect flow, and the weir actually acts as a flow divider. Possible repairs to this situation would be (a) installation of longitudinal peaked stone toe protection between the last two weirs and continuing 100 ft downstream from the root of the last weir, (b) ensuring that the stone in the weir key is flush with the surrounding bank, and (c) allowing the point bar to reshape after project installation, then building one or two structures on the point bar that would correctly aim high-flow currents toward the weir field at the downstream end of the bend.

Very little rock settling (weir subsidence) has been observed. The only weirs that appear to have been damaged by high flows or debris are two or three weirs in Reach 12 where the crest elevation near the bank end is lower than the crest elevation at the stream end. Weir performance does not appear to have been compromised and the bank is stable in this area. Scour holes at the stream ends of the weirs have appeared stable, but did not deepen enough to launch rock from the end of the weir.

Weir angle is extremely important. The operating theory for bendway weirs was that flow could be captured and redirected by the weirs at the upper end of the bend, controlled through the bend, and aimed in the direction needed by the last weir in the bend. This theory has been essentially confirmed during the first year after project completion. Weirs incorrectly angled near the center of the weir field (Reach 11) do not appear to compromise performance. Weirs incorrectly angled near the downstream end of the bend or in an area of high flow concentration, as in Reach 9, can lead to bank erosion problems.

This project has shown that the thalweg of the stream can be aimed perpendicular to the axis of the last weir of a set. This is critically important

with the extremely short crossing lengths found in this demonstration reach. Also the weirs at the upstream end of the bend must be angled correctly to capture and align the flow for the next weirs in the bend.

One of the stream features identified by ARS as being needed for successful aquatic habitat improvement or restoration was the presence of woody debris. Woody debris correctly positioned within a weir field will stay in place even with repeated out-of-bank flows. The woody debris that was either in place or moved into place has remained in position. The debris that collected was about midway down the length of the bendway weir and did not appear to adversely affect bank stability. First year sampling results (June and October 1994) show that twice as many fish were found in areas using bendway weirs compared with willow post or stone toe protection areas.

A single correctly positioned weir can have advantages over what would normally be expected. In Reach 4 a single weir 27 ft in length has changed the direction of flow and controls the stream for a length of 150 to 200 ft in the downstream direction. In Reach 10 a single weir caused 2 to 3 ft of aggregation upstream of the weir covering an area 20 to 30 ft wide and approximately 100 ft long.

Willow posts

The willow posts were planted when dormant starting around 1 February 1994 and ending on 6 March 1994. One serious problem was that the willow post holes were not adequately backfilled after post installation. The Vicksburg District contract for the installation of the willow posts stated that "...a cohesive bond is formed between the post and the natural ground." This statement was apparently open to interpretation, but for whatever reason, many of the willow post holes were not backfilled as desired. It is suggested that future contract language be specific in requiring that, after post installation, the auger holes must be backfilled to ground level, tamped, and watered.

Approximately 3 weeks after completion of the willow post planting, a high-flow event inundated the project. This flow was beneficial because it backfilled the augured holes. However, this flow was also detrimental, as most of the initial flush (branches that sprout immediately after planting from stored energy in the post and before any root mass has established) was broken off at the growth node. This high-velocity flow stressed the willows, many appeared to die, and individuals that recovered exhibited a reduced growth rate. However, in areas where willows were shielded from this flow (sections of Reaches 4 and 10), recovery and growth were markedly better than in the rest of the project.

Overall the performance of the willow posts as bank protection is mixed. It appears that virtually all of the posts sprouted in the spring, but many were subjected to intense stress from flow and debris, while others were damaged by beavers. Several bends or portions of bends appear to have been stabilized with the growing willow posts, while other bends have been stabilized only to

the extent that numerous willow posts are present and providing a measure of stability even though the posts appear to be dead. However, as late as October 1994 many seemingly "dead" posts were developing new growth at or below ground level. In Reach 13 one section of bank has had most of the posts uprooted and transported away.

Even with a low survival rate (42 percent living after 1 year), many areas of the stream protected by the willow post method appear stable with abundant volunteer growth. While survival rate is certainly one measure of project success, long-term stability will depend on the initial (post planting) stability of the bank and the ability of native plants to colonize the bank. The close spacing and deep planting used in this project appear to have worked well.

Willows split during installation or by impact from debris died during the first growing period. Again, future contract language should state that if a willow is split during installation, the post should be removed and a healthy willow post installed in its place.

Damage by insects was minimal, probably limited to 10 to 20 posts. However, beavers did chew on the installed willow posts and the 1/2- to 3/4-in.-diameter young branches. Very small branches do not appear to be of interest. While the beaver activity has been fairly intense in at least two reaches, project stability has not been compromised. In many cases the willow sprouts new growth and flourishes.

If bank erosion causes a willow post that does not have a well-established root system to become disconnected from the bank and located in the stream, that willow will eventually die. Typically a willow can be inundated only for about 3 months. In some areas more than the first row of willows has become inundated. In these areas use of a minimum stone toe protection streamward of the first row of willows would be required. Other possible methods of toe protection that could be considered are branch matting, geotextile, seeded or vegetated mat, coconut rolls, live fascines, or anchored brush or trees. Also, willows need to be planted in areas where water will not puddle. If water puddles around the willows, they will eventually die.

In some slide areas the willows held well, with little or no movement or rotation. However, even with the willows planted to a depth of 10 ft, geotechnical bank failure was not prevented in at least two locations (sections of Reaches 4 and 10). In two areas (Reaches 10 and 13) the posts were carried off during subsequent high flows.

Results with willows planted landward of longitudinal peaked stone toe protection were mixed. If the willows were planted on a sloped bank at an elevation greater than the crest of the stone protection, overall survival and growth rates were good (Reach 5). If the willows were planted in the depositional area landward of the stone protection at an elevation lower than the crest of the protection, water puddled and drowned the willows (Reach 14). If stone toe protection is used streamward of willow posts, the posts must be

installed on a sloped and compacted bank above the stone protection crest elevation. If conditions are such that this cannot be accomplished, the stone protection needs to be more like bank paving (laid on existing or reworked slope) with the willows upslope.

Conclusions

All bank stabilization measures were installed by March 1994; therefore, sufficient time has passed to make an initial evaluation of project performance. At least four out-of-bank flow events have occurred since installation, so the observed effectiveness of these bank protection methods should be indicative of future performance (even under adverse conditions). While problems with construction of the bendway weirs and installation of the willow posts created some initial problems, the project results after the first year have been satisfactory, with most areas appearing stable and maturing quickly. In addition, aquatic and stream corridor habitat has been improved.

Proposed Bioengineering Applications for Harland Creek

Introduction

The objective of this effort was to design five bioengineering sites for installation and evaluate the performance of such sites within the DEC watersheds. The sites were surveyed and conceptual designs prepared by CSU (Watson, Abt, and Thornton 1994). That report presents detailed background information concerning design, construction, and inspection of numerous bioengineering techniques; typical drawings for bioengineering techniques; and a topographic survey of each site.

Bioengineering combines biological elements with engineering design principles. The requirements for both must be considered when planning and designing bioengineered measures. For example, engineering requirements may dictate highly compacted soil for fill slopes, whereas plants prefer relatively loose soil. Using a sheepsfoot roller for compaction is a solution that would integrate biological and engineering requirements because the compactor compacts the soil, but also allows plant establishment in the resulting depressions in the slope. Differing needs can generally be integrated through creative approaches and compromises in planning and design.

Bioengineering sites

After consideration was given to various DEC streams for installation of bioengineering techniques, it was eventually decided to make the application on Site 1 of Harland Creek (Chapter 3). Surveys were made of five proposed sites

along Harland Creek in June 1994. Based on the surveys, site visits, and the compiled background information, a combination of bioengineering treatments were developed. A preliminary design was reviewed by WES and the Vicksburg District and a field review of the proposed plans was completed prior to submission of the final plans.

The site numbers presented here should not be confused with the annual monitoring site numbers presented elsewhere in this report. To minimize any confusion and to associate the sites and plans presented in the following paragraphs, the proposed bioengineering sites will have a "BIO" suffix added to the site number.

Site 1-BIO. The recommended plan for Site 1-BIO is to construct 350 ft of vegetated toe riprap along the toe of the left bank between riprap placed at the county road bridge and longitudinal toe riprap placed by the Vicksburg District. Landward of the vegetated toe riprap, two rows of willow posts will be installed at a spacing of 3 ft on center. The top of fill should slope at approximately 1V on 2H to prevent ponding behind the stone. All disturbed areas should be seeded in accordance with standard criteria.

Vegetated toe riprap is a combination of rock and live branch cuttings used to stabilize and protect the toe of steep slopes, as shown in Figure 70. Vegetated riprap is not intended to resist large lateral earth pressures, and is for primary use in resisting impinging flow. Such a system is appropriate at the toe of a slope to resist erosion and to establish vegetation. Live cuttings should have a diameter of 1/2 to 1 in. and be long enough to reach beyond the rock structure into the fill or undisturbed soil behind. The riprap toe section should be installed using 1 ton of stone per linear foot.

The vegetated toe riprap should be installed in vertical lifts. A third of the toe riprap should be placed and then a layer of live willow branch cuttings. This layer should be covered with a layer of soil 4 to 6 in. thick followed by another layer of riprap, a layer of soil and branches, followed by a final layer of riprap. The top of fill material behind the riprap should slope gently toward the stream to prevent ponding behind the stone, which can inhibit vegetation. The live branch cuttings should be oriented perpendicular to the slope contour with growing tips protruding slightly from the finished rock wall face.

The application of willow posts in Site 1-BIO is very similar to applications installed in Harland Creek discussed earlier in this chapter. The willow post technique consists of harvesting sections of trunk 10 to 12 ft in length and 3 to 6 in. in diameter after the tree is dormant, and inserting the pole into augured holes 6 to 8 ft deep spaced at regular intervals along an eroding stream bank. Willow poles perform several immediate and long-term functions in erosion control, earth reinforcement, and mass stability of slopes. The streambanks are strengthened by soil arching and by pinning of possible failure surfaces. The bank materials are mechanically reinforced by willow root growth. Stream velocities along the bank are reduced, thereby reducing the potential for bank material to be eroded and transported downstream. The willows provide a

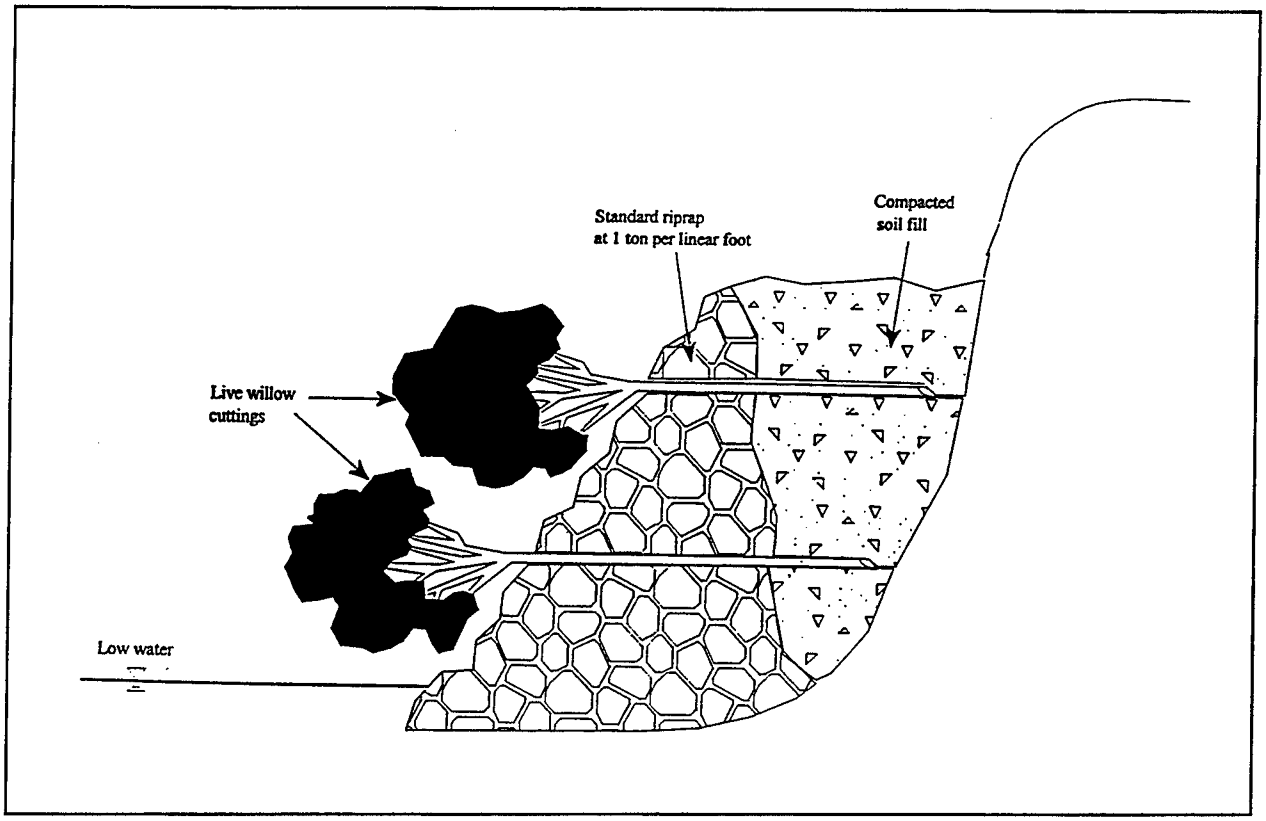

Figure 70. Vegetated toe riprap

microclimate for colonization by other species, and debris is trapped. Soil block failures from upslope areas are retained within the willow posts. The posts also provide a sound anchor for other bioengineering techniques (such as fascines) placed along the toe of the bank, cribwall support, or use of large woody debris attached at the water edge.

Willow posts should be taken from trunk sections of willow trees with branches removed. Post diameter should be a minimum of 3 in. at the butt end. Minimum post length should be 10 ft. Native willows in good condition and with bark relatively undamaged should be used. Bark should not be damaged during installation. Posts should be harvested during the dormant season, soaked in water, and planted within 48 hours. Tops of the posts should be marked during harvest to ensure planting right side up. Posts should be planted in augured holes at least 8 ft in depth, and the post holes should be backfilled to ensure that no voids exist, that the post is fully in contact with the soil, and the butt is in contact with the bottom of the post hole. The holes should be augured in a grid spacing of 3 ft on centers, i.e., between rows and between posts in a single row. An 8-in.-diameter auger is recommended. No more than 4 ft of the post should be above ground following installation. Grass seeding of the site is required following installation.

Site 2-BIO. The recommended plan is to construct a 150-ft-long cribwall to provide a smooth connection with two existing sections of longitudinal stone toe dikes. The crib will be 9 ft thick, consisting of three cells 3 ft thick as measured from streamside to landward side using four rows of willow posts installed at 3 ft on center. The controlling alignment of the crib is the streamside face, which must smoothly tie from the upstream to the downstream existing longitudinal riprap toe. If the streamside alignment does not permit a full thickness of 9 ft without excavating the existing bank, the thickness of the crib may be decreased to no less than 3 ft. The crib thickness will, in all cases, occupy from the streamside alignment to the existing bank up to a maximum thickness of 9 ft. The crib will be approximately 6 ft in height, which can be accomplished in a stair-step arrangement. Total height is measured from the streamside.

The crib will be filled with riprap up to 6 in. above the approximate low-water surface elevation, the streamside cell will be filled with burlap bags of soil, and alternating layers of soil and branches will be used to fill the remaining cells up to the total height. Bags will be placed in the streamside cell to allow branches to protrude through the riprap and overhang the stream.

Two additional rows of willow posts will be installed landward of the live crib, and fill will be placed to the elevation of the top of the crib and sloped toward the stream for drainage. All disturbed or fill areas will be seeded in accordance with standard criteria.

General guidance for a live cribwall consists of a hollow, boxlike interlocking arrangement of untreated log members. Construction uses live willow members (same as willow posts) assembled in log-cabin fashion as shown in Figure 71. The structure is filled with suitable backfill material and layers of live branch cuttings, which root inside the crib structure and extend into the slope. After the live cuttings root and become established, the developing vegetation gradually takes over the structural functions of the wood members.

Cribwall structures can be used in areas at the base of a slope where a low wall may be required to stabilize the toe of the slope and reduce steepness. Cribwalls are particularly useful where space is limited and a more vertical structure is required. Cribs provide immediate protection from erosion, while established vegetation provides long-term stability. These structures are not designed for or intended to resist large, lateral earth stresses and should be constructed to a maximum overall height of 4 ft. However, the structures may be constructed in a stair-step fashion, with each successive course of timbers set back 6 to 9 in. toward the slope face from the previously installed course. If the system is built on a smooth, evenly sloped surface, the cribwall face should be tilted back or battered.

The live branch cuttings should be 1/2 to 2 in. in diameter and long enough to reach the back of the wooden crib structure. Willow posts used in the cribwall should of the same size and installed in accordance with the guidelines

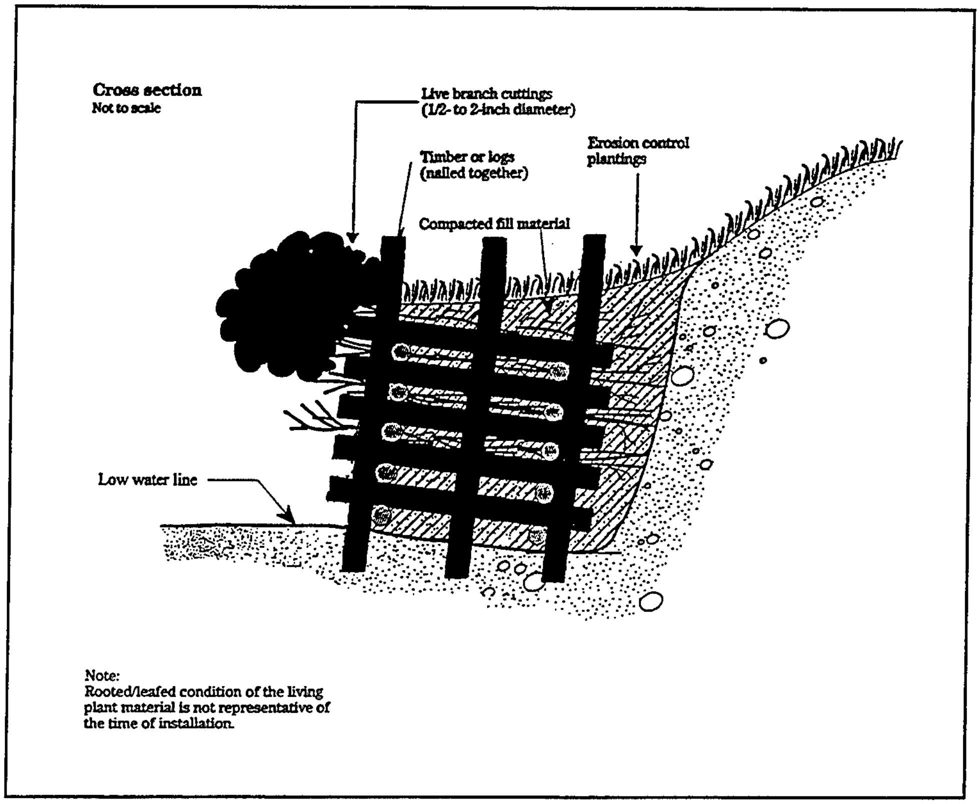

Figure 71. A live cribwall using live willow posts and cross members

described previously. Large nails or reinforcement bars are required to secure the logs or timbers together. The cribwall construction starts with the installation of three rows of willow posts on 2- to 3-ft centers, starting at the lowest point of the slope and excavating loose material between the posts until a stable foundation is reached. The back of the stable foundation is then excavated slightly deeper than the front to add stability to the structure. The first course of logs or timbers is placed at the front and back of the excavated foundation, against and inside of the lower and higher row of willow posts. Then the next course of logs or timbers is placed at right angles on top of the previous course, overhanging the front and back by 3 to 6 in. Each course of the live cribwall is placed in the same manner and nailed to the preceding course with nails or reinforcement bars. Live branch cuttings should be placed at each course to the top of the cribwall structure with growing tips extending out of the cribwall. Each layer of branches should be followed with a layer of compacted soil to ensure soil contact with the live branch cuttings. Cuttings should reach to undisturbed soil at the back of the cribwall with growing tips protruding slightly beyond the front of the cribwall.

Site 3-BIO. At Site 3-BIO, four rows of willow posts will be installed on the right bank for approximately 300 ft and tying into the existing upstream longitudinal stone toe revetment. The willow posts will be installed at a spacing of 4 ft on center. The lower row of willow posts will be placed

approximately 1 ft above low-water surface elevation. A willow fascine will be installed on the landward side of the first row of willow posts.

Riverward of the first row of willow posts, downed willow trees with limbs and with the tree butts pointed upstream will be anchored along the toe of the bank and cabled to the first row of willows. Trees must be bushy, at least 12 ft in length, and with a butt diameter of 6 to 12 in. Trees must overlap by at least 3 ft, and each tree will be anchored to at least two willow posts at a spacing of approximately 6 ft using steel cable and cable clamps. The trunk of the tree should be in contact with the stream bottom.

The point bar on the opposite bank from Site 3-BIO will be excavated to provide a channel approximately equal in width to the existing channel at the upstream limits of the site. All disturbed bank areas will be seeded in accordance with standard criteria.

Live fascines are long bundles of branch cuttings bound together into sausage-like structures and placed in shallow contour trenches, as shown in Figure 72. When cut from an appropriate species and properly installed with live and dead stout stakes, cuttings will root and begin to stabilize slopes. This system, installed by a trained crew, should not cause much site disturbance. Fascines placed at an angle to the contour can act as drains on wet slopes. Fascines are an effective stabilization technique for slopes and may be suitable for toe protection. They protect slopes from shallow slides (1 to 2 ft deep) and immediately reduce surface erosion or rilling. They are also suited to steep, rocky slopes, where digging is difficult. Fascines are capable of trapping and holding soil on the face of the slope, which transforms a long slope into a series of shorter slopes. They enhance establishment of vegetative cover by creating a microclimate conducive to plant growth.

Fascines are constructed from cuttings of species, such as young willows, privet, or river birch, which root easily and have long, straight branches. The cuttings are tied together to form live bundles that vary in length from 5 to 30 ft or longer, depending on site conditions and limitations in handling. Normally, the fascine bundles are assembled on X-frames using trimmed willow poles, branches, or tops. The completed bundles should be 6 to 8 in. in diameter, with all of the growing tips oriented in the same direction. The cuttings are staggered so that the tops are evenly distributed throughout the length of the uniformly sized live fascine. Live stakes used to anchor the fascines should be 2-1/2 ft long in cut slopes and 3 ft long in fill slopes. String used for bundling should be untreated twine. Dead stout stakes used to secure the live fascines should be 2-1/2-ft-long, untreated, 2 by 4 lumber. Each length should be cut again diagonally across the 4-in. face to make two stakes from each length. Only new, sound, unused lumber should be used, and any stakes that shatter upon installation should be discarded.

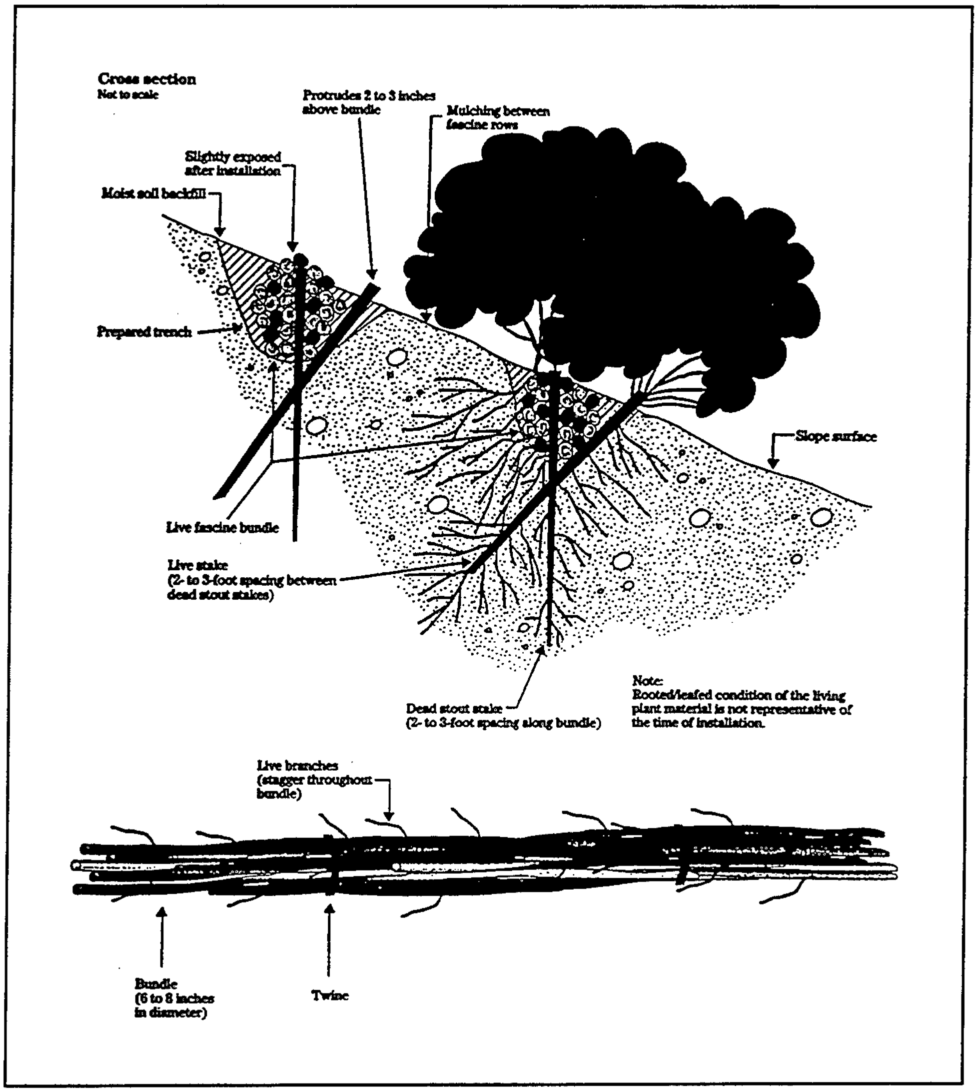

Figure 72. Live fascine installation (after SCS 1992)

Once the live fascine bundles and live stakes have been prepared, installation can take place. Beginning at the base of the slope, a trench should be dug along the contour just large enough to contain the live fascine. The trench will vary in width from 12 to 18 in., depending on the angle of the slope to be treated. The depth of the trench will be 6 to 8 in., depending on the size of the final individual bundles. Once the live fascines are placed in the trench, the dead stout stakes are driven directly through the live fascine every 2 to 3 ft along its length, leaving the top of the stakes flush with the installed bundle. Extra stakes should be used at connections or bundle overlaps. Live stakes are generally installed on the downslope side of the bundle. Drive the live stakes below and against the bundle between the previously installed dead stout stakes. The live stakes should protrude 2 to 3 in. above the top of the live fascine. Moist soil should be then be placed along the sides of the live fascine. The top of the fascine should be slightly visible when the installation is completed. Fascines can be installed at intervals on the contour up the slope to resist

erosion. Installations angled slightly downslope provide drainage routes for slopes with excess water.

Site 4-BIO. A vegetated riprap toe was designed for this site, which is on the right bank. The downstream extent of the vegetated riprap toe will tie into the existing longitudinal riprap toe and extend upstream 425 ft. The upstream extent of the vegetated toe riprap is an unstable slide area. The slide area appears to be very unstable, and no excavation or vehicle traffic should be permitted on the site.

Landward of the vegetated toe riprap, branchpacking will be installed. The branchpacking will tie from the elevation of the fill landward of the riprap to the right top bank. If the slope of the branchpacking fill required to match the top bank is greater than a 1V:2H slope, the branchpacking will be placed at the 1V:2H slope up to the intersection with the existing bank. Intervals between vegetation, composed of willow and privet branches, will be no more than 3 ft vertically. A combination of willow and privet branches will be used in the branchpacking. Along the left bank near the downstream limits of the site, a debris pile will be removed from the stream. All disturbed or fill areas will be seeded in accordance with standard criteria.

Branchpacking consists of alternating layers of live branch cuttings and compacted backfill to repair small localized slumps and holes in banks, as shown in Figure 73. Branchpacking is effective in earth reinforcement and mass stability of small earthen fill sites producing a filter barrier that reduces erosion and scouring. Due to the method of branchpacking construction, it provides an immediate soil reinforcement. It can also be used to repair holes in earthen embankments, other than dams, where water retention is required.

Branchpacking is made from live branch cuttings ranging from 1/2 to 2 in. in diameter. The cuttings should be long enough to touch the undisturbed soil at the back of the trench and extend slightly from the rebuilt slope face. The willow poles should be 5 to 8 ft long and from 3 to 6 in. in diameter, depending upon the depth of the particular slump or hole. Starting at the lowest point, the willow poles should be installed on 3-ft centers. Then a layer of living branches 4 to 6 in. thick is placed in the bottom of the hole, between the vertical willow poles. Branches should be placed in a criss-cross configuration with the growing tips generally oriented away from the slope face. Some of the basal ends of the branches should touch the back of the hole or slope. Subsequent layers of branches are installed with the basal ends lower than the growing tips of the branches. Each layer of branches must be followed by a layer of compacted soil to ensure soil contact with the branch cuttings. The final layer should match the existing slope. Branches should protrude only slightly from the filled face. Once installation is complete, the soil should be kept moist or moistened to ensure that live branches do not dry out.

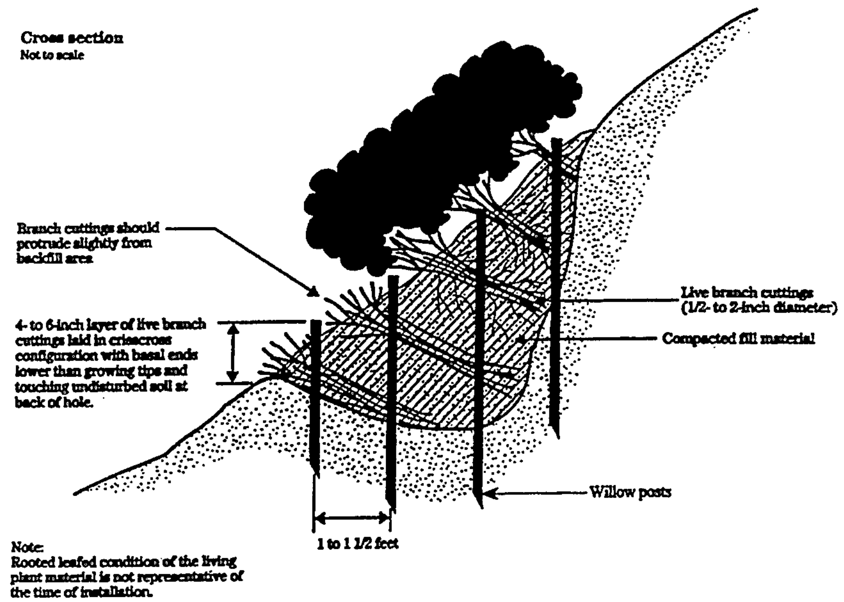

Figure 73. Branchpacking can be used to fill gullies or bank irregularities (after Schiechtl 1980)

As plant tops begin to grow, the branchpacking system becomes increasingly effective in retarding runoff and reducing surface erosion. Trapped sediment refills the localized slumps or holes, while roots spread throughout the backfill and surrounding earth to form a unified mass. Branchpacking may also be effective in repairing streambank gullies.

Site 5-BIO. This site is adjacent to and immediately upstream of Site 4-BIO, and comprises the unstable slide area and the right bank upstream of the slide to the upstream right bank point bar. The point bar begins approximately 300 ft upstream of the slide area. Five rows of willow posts will be installed at a spacing of 3 ft on center into the loose material at the toe of the slide beginning approximately 1 ft above the low-water elevation. A cribwall will be installed upstream of the slide area and continue 300 ft upstream. Three rows of willow posts will be used in installation of the cribwall, for a crib thickness of 6 ft. The crib will be filled using riprap up to the low-water surface elevation, and with alternating layers of willow branches and soil for the remainder of the fill. Soil in the streamside cell will be contained in burlap bags to resist erosion of the soil. The five rows of willow posts on the slide area will tie smoothly between the downstream vegetated riprap toe of Site 4-BIO and the upstream cribwall.

The slide area is very unstable. No excavation of the slide except for augering the post holes should be permitted. All fill areas should be seeded in accordance with standard criteria.

Recommended construction phasing

A three-phase construction process should be considered for the installation of the bioengineering techniques on the five sites on Harland Creek. Phase 1 would be initial construction, Phase 2 would be the first year of the establishment period, and Phase 3 would be the second year of the establishment period. The establishment period for 2 years permits replacement of dead or damaged plant materials, spraying for insect infestations, or other undesirable conditions. A plan for planting a diverse mix of woody vegetation will be prepared by CSU following construction. This plan is suggested for installation during Phase 2 construction. Local maintenance would be expected after the final inspection that is recommended at 2 years following the initial construction.

The following schedule for inspections and quality control maintenance of the sites is presented:

a. Select plant species for conformance to requirements.

b. Locate and secure source sites for harvesting live cuttings or commercial procurement.

c. Define construction work area limits.

d. Fence off sites requiring special protection such as special habitat or areas of cultural significance.

e. Complete and inspect the following preparations:

(1) Layout

(2) Excavation and systems excavation.

(3) Bench size, shape, and angle.

(4) Preparation of site; i.e., clearing, grading, and shaping.

(5) Disposal of excess gravel, soil, and debris.

(6) Depth of excavation.

(7) Vegetation to be removed or preserved.

(8) Stockpiling of suitable soil and rock.

f. Inspect each system component at every stage for:

(1) Angle of placement and orientation of the live cuttings.

(2) Backfill material, rock, and stone material.

(3) Fertilizer including method and quantity applied.

(4) Lime including method and quantity applied.

(5) Preparation of trenches or benches in cut and fill slopes.

(6) Staking.

(7) Pruning.

(8) Stock handling and preparation.

(9) Soil compaction.

(10) Watering.

g. Ensure that proper maintenance occurs during and after installation.

h. Inspect daily for quality control.

(1) Check all cuttings, remove unacceptable material, and use fresh stock for replacement installations.

(2) Continuously check all items in the preconstruction and construction inspection lists.

(3) Inspect the plant materials storage area when it is in use.

i. Interim inspections should be made after the bioengineering measures have been installed.

(1) Inspect biweekly for the first 2 months. Inspections should note insect infestations, soil moisture, and other conditions that could lead to poor survivability. Immediate action, such as the application of supplemental water or insect spraying, should be taken if conditions warrant.

(2) Inspect monthly for the next 6 months. Systems not in acceptable growing condition should be noted and, as soon as conditions permit, should be removed from the site and replaced with materials of the same species and sizes as originally specified.

(3) Needed reestablishment work should be performed every 6 months during the initial 2-year establishment period. This will usually consist of replacing dead material.

(4) Extra inspections should always be made during periods of drought or heavy rains.

j. A final inspection should be held 2 years after the installation is completed.

(1) Healthy growing conditions in all areas refer to overall leaf development and rooted stems defined as follows:

Live stakes	70-100 percent growing
Live fascines	20-50 percent growing
Live cribwall	30-60 percent growing
Branchpacking	40-70 percent growing
Willow posts	60-80 percent growing
Vegetated toe riprap	50-80 percent growing

(2) Growth should be continuous with no open spaces greater than 2 ft in linear systems. Spaces 2 ft or less will fill in without hampering the integrity of the installed living system.

Maintaining the system

After inspection and acceptance of the established system, maintenance requirements should be minor under normal conditions. Maintenance generally consists of light pruning and removal of undesirable vegetation. Heavy pruning may be required to reduce competition for light or stimulate new growth in the project plantings. In many situations, installed bioengineering systems become source sites for future harvesting operations. The selective removal of vegetation may be required to eliminate undesirable invading species that should be cut every 3 to 7 years.

More intensive maintenance will occasionally be required to repair problem areas created by high-intensity storms or other unusual conditions. Site washouts should be repaired immediately. Generally, reestablishment should take place for a 2-year period following the completion of construction and consist of the following practices:

a. Replacement of branches in dead unrooted sections.

b. Soil refilling, branchpacking, and compacting in rills and gullies.

c. Insect and disease control.

d. Weed control.

Gullies, rills, or damaged sections should be repaired through the use of healthy, live branch cuttings preferably installed during the dormant season. If the dormant season has elapsed, the use of rooted stock may be considered.

Summary of proposed bioengineering sites

The five sites proposed for application of bioengineering techniques include installation of methods installed in other portions of the DEC Project and installation of techniques installed in other similar projects. The particular methods proposed are based on those past practices and incorporate lessons learned from those practices. The proposed sites and methods are for Harland Creek and include the following:

Site 1-BIO. A 350-ft-long vegetated toe riprap along the toe of the left bank between existing riprap and two rows of willow posts landward of the vegetated toe riprap.

Site 2-BIO. A 150-ft-long cribwall between two existing sections of longitudinal stone toe dikes with two additional rows of willow posts installed landward of the live crib.

Site 3-BIO. Four rows of willow posts on the right bank for approximately 300 ft tying into the existing upstream longitudinal stone toe revetment. A willow fascine will be installed on the landward side of the first row of willow posts. Riverward of the first row of willow posts, downed willow trees will be anchored along the toe of the bank and cabled to the first row of willows. The point bar on the opposite bank will also be excavated.

Site 4-BIO. A vegetated toe riprap on the right bank extending into the existing longitudinal riprap toe. Landward of the vegetated toe riprap, branchpacking will be installed. Along the left bank near the downstream limits of the site, a debris pile will be removed.

Site 5-BIO. Five rows of willow posts installed in the unstable slide area and a 300-ft-long cribwall installed upstream of the slide area.

7 Technology Transfer

As stated earlier, technology transfer is an important part of the DEC program and will be given high priority at WES during the monitoring program. During FY 1994 the results of the DEC monitoring work and the various designs that were developed and installed were provided to several agencies and organizations. These applications were on streams with problems similar to the DEC streams and provided the various agencies with potential solutions that would not have been available to them without the DEC Project. The various technology transfer efforts completed in FY 1994 are discussed in this chapter.

A video on channel degradation was completed and provided to the Vicksburg District. This video presents the various aspects relative to channel degradation including erosion and shoaling processes and methods for channel stabilization. There have been numerous requests for the video, and it has proved itself as an excellent training tool in workshops and training classes dealing with bank stabilization. Dr. William Rahmeyer and several other engineering professors from Utah State University have used this and another DEC video for educational purposes.

In February 1994, representatives from WES attended the International Erosion Control Association Conference and Trade Exposition in Reno, NV. A display booth presenting the WES DEC monitoring effort and overall DEC project was manned by the representatives during the week. Significant interest was shown by conference and exposition attendees relative to the monitoring effort, and there were numerous requests for DEC information.

The ARS office in Oxford, MS, requested and was provided DEC database information on riser pipes. The ARS used that information in habitat and water quality studies being conducted around riser pipes.

A workshop on the use of the CASC2D watershed model was conducted in Memphis, TN, in June 1994. The workshop included presentation of the model methodology, model input requirements, and a case study for potential model users.

Mr. Don Roseboom of the Illinois State Water Survey requested assistance in a bendway weir design for bank stabilization on Crow Creek near Peoria, IL. A field reconnaissance and proposed design by WES personnel were also provided for Senachwine Creek near Peoria. Mr. Roseboom originally assisted WES personnel on the willow post design used as a basis for the applications on Harland Creek.

Mr. Wayne Kinney of the Edwardsville, IL, office of the SCS also requested assistance in a bendway weir design for Cahokia and Wood Creeks near East St. Louis, IL.

Representatives of the U.S. Army Engineer District, New York, toured the Harland Creek willow post and bendway weir sites. That office was considering possible application of those types of bank stabilization methods to streams in their area.

8 FY 1995 Work Plan

This work plan presents the work areas and reporting activities for the DEC Project Monitoring Program to be conducted by WES in FY 1995.

The purpose of the DEC Project Monitoring Program is to evaluate and document watershed response to the implemented project features. One major goal of the DEC Project is to reduce sediment yield to the Yazoo River. Therefore, a major objective of the monitoring program is to determine the effectiveness of DEC Project features in reducing sediment yield. Documentation of watershed responses to DEC Project features will allow the participating agencies a unique opportunity to determine the effectiveness of existing design guidance for erosion and flood control in small watersheds.

This work plan proposes 11 technical areas for the DEC monitoring program that will effectively monitor the major physical processes of erosion. The following areas are to be monitored and/or addressed:

a. Stream gauging.

b. Data collection and data management.

c. Hydraulic performance of structures.

d. Channel response.

e. Hydrology.

f. Upland watersheds.

g. Reservoir sedimentation.

h. Environmental aspects.

i. Streambank stability.

j. Design tools.

k. Technology transfer.

WES is proposing significant activities in all technical areas except environmental aspects. A major effort for FY 1995 will be in project evaluation to date and the development of design tools for spacing of grade control structures. Additionally, the past practices and designs of bank stabilization on the DEC streams will be documented and published as design criteria. It is understood that these published criteria are of an interim nature and subject to modification as additional data are obtained in the future and other more recently constructed methods are monitored and evaluated.

The following is a general description of the work to be performed in the ten technical areas by WES. The specific work tasks discussed in each work area should be viewed as a starting point for planning the FY 1995 monitoring program. It is anticipated that the monitoring program will need to be modified as data are collected and analyzed and as new and different areas of concern develop. To accomplish this, the Hydraulics Laboratory will work closely with the Vicksburg District personnel and will schedule quarterly review sessions with the Vicksburg District. This will allow the monitoring program to be adjusted as necessary to meet the needs of the DEC Project.

Data Collection and Data Management

The purpose of the Data Collection and Data Management work area is to assemble, to the extent possible, all data that have been collected to date in the DEC Project, and to develop an engineering database/GIS that is continually updated as new data are collected and analyzed. The database resides on an Intergraph computer platform, and access to the database is currently through Intergraph workstations. The database contains aerial and satellite photography, watershed maps with DEC structure reports, USGS digital elevation grids, USGS quadrangle maps, soil type grids, and land use grids. The database now contains the information needed to use the small watershed hydrology design tool for the hydraulic design of riser pipes. The hydraulic, sedimentation, and geometric survey data being collected in the monitoring program will be added to the database, along with various project feature designs and specifications, trip reports and field observations, and an index of study reports by others. As data are placed into the database, it will be used by WES to effectively conduct tasks in the monitoring program such as channel response evaluations and sediment yield reduction studies and should also become increasingly useful to the Vicksburg District for engineering activities related to watershed erosion control in the DEC Project. The database has been modified to allow access to the database for GRASS users at the ARS and SCS.

For FY 1995, the placement of collected data into the engineering database will continue, including WES, Vicksburg District, CSU, ARS, and SCS data. Historical data will continue to be added as necessary. Database maintenance,

software updates, and user support will be conducted as necessary. Relative to data collection, a major change from the past practices of obtaining stage or crest gauge data with corresponding, but limited, discharge data will be undertaken: acoustical discharge measurement equipment will be installed on 10 of the DEC monitoring sites without structures. Limited discharge ratings have been obtained at these sites because of the difficulty of personnel being at the site during major flow events. This equipment should eliminate that problem. The major FY 1995 tasks are as follows:

a. Continue stage data collection at established gauging stations.

b. Continue discharge measurements at established stations and add new acoustic discharge instruments.

c. Continue quality control processing of stage data.

d. Update stage-discharge rating curve for each established gauging station.

e. Develop discharge hydrographs for the 23 long-term monitoring sites using stage, discharge gauging, and survey data.

f. Continue routine database maintenance, updates, and support.

g. Continue to build engineering database as new data become available.

h. Add FY 1992 to FY 1994 long-term monitoring site surveys.

Hydraulic Performance of Structures

Data collection and data analysis will be a continuing effort in FY 1995 for selected drop structures for use in developing discharge relationships for those DEC streams. A biennial visual inspection program for all grade control structures in the DEC Project has been established, and the first-year inspection was completed in FY 1993. A biennial visual inspection to include selected bank stabilization structures was initiated in FY 1994.

The biennial visual inspection of grade control structures will be repeated, and the biennial inspection of selected bank stabilization structures, which was initiated in FY 1994, will be completed. The inspection of the drop structures will be expanded somewhat to include channel thalweg surveys approximately 1,500 ft upstream and 1,000 ft downstream of each structure. The major FY 1995 tasks will be to conduct and document visual inspection of all drop structures and selected bank stabilization structures in the DEC watersheds.

Channel Response

The channel response monitoring is being directed toward channel sedimentation. The 23 sites where structures exist or are anticipated will continue to be extensively monitored. Channels upstream and downstream of the structures will continue to be monitored for cross-section changes, thalweg changes, berm formation, bank failure, and vegetation development. Five selected sites where no structures are planned, which serve as control and will assist in the evaluation of the channel response to structures, will also continue to be monitored. Structures and channels at selected long-term monitoring sites have been instrumented for stage and discharge by WES. Suspended sediment concentrations are being measured at other locations by USGS. Bed-material samples at the 23 long-term sites are being collected by CSU. HEC-6 and the computer program SAM will be used to predict the stability of channels monitored by this work effort.

For FY 1995, work in the channel response technical area will include continued data collection and analysis at the 23 long-term monitoring sites. The Hotophia Creek sediment reduction study will be completed. The objective of the study will be to quantify the reduction in sediment yield as a result of the DEC Project. Three DEC watersheds, Long and Batupan Bogue and a third to be determined later, will be used to determine short- and long-term effects of previous DEC work on these streams. The major FY 1995 tasks are as follows:

a. Conduct long-term monitoring at 23 selected sites.

b. Complete Hotophia Creek sediment reduction study.

c. Conduct long-term predictive modeling for three DEC watersheds.

Hydrology

Rainfall runoff provides the energy to sustain erosion processes. The ability to measure rainfall and compute runoff accurately is crucial in the design of stable flood-control channels. Accurate flow rates are needed to design functional project features properly and maintain stability in the channel system.

In FY 1994 hydrologic models (CASC2D) for four DEC watersheds were developed. In FY 1995 hydrologic models will be developed for use on the channel response effort on Long and Batupan Bogue watersheds and to support the CSU long-term monitored sites. The application of CASC2D, with improved channel routing, continuous simulations, GRASS GIS linkage, and watershed erosion, will allow basinwide simulations over a long period of time (i.e., 20 years). With CASC2D linked to a GIS, different types or numbers of

hydraulic structures can be tested for sediment yield reduction. Also, predicted changes in land use can be tested for future impact on sediment yield. The second year of a contract with CSU to develop a procedure for the development of the effective discharge for each of the 23 DEC monitoring sites will be continued. The major FY 1995 tasks are as follows:

a. Compute 1993 and 1994 discharges for the long-term monitoring sites.

b. Construct CASC2D models for the DEC watersheds in support of channel response studies on Long and Batupan Bogue watersheds and in support of CSU monitored sites.

c. Compare radar rainfall (NexRAD) overlays for the Goodwin Creek watershed with the observed 1994 rainfall.

d. Continue development of procedures for computing the effective discharge at the 23 DEC monitoring sites.

Upland Watersheds

The purpose of the Upland Watershed technical area is to determine if there is a measurable change in the quantity of sediment being transported from each watershed for the next 5 years. Data that have already been collected by the USGS and ARS for the past 5 years will be analyzed to serve as the basis for future comparisons. Numerical modeling of the sediment runoff from the watersheds will be incorporated into the data analysis and interpretation process. Sediment production from two or three active gullies will be analyzed by comparing surveys made prior to the design of drop pipes and the survey made just prior to construction of the drop pipes.

In FY 1994, WES tested the applicability of the sediment yield model, GISSRM, on the Goodwin Creek watershed. The model was calibrated and pre- and post-comparisons of sediment yield were made. In FY 1995 validation of the GISSRM model will be continued. The major FY 1995 tasks are as follows:

a. Continue separation of sediment yield into basic sediment sources (land surface, gully, bank, and bed erosion).

b. Continue validation of the sediment yield model, GISSRM, to Goodwin Creek.

c. Continue testing selected scenarios to determine future conditions for sediment yield.

Reservoir Sedimentation

The major sources of reservoir sediment deposits are upland erosion, erosion of the channel banks, and erosion of the channel bed. The reduction of the inflowing sediment load is being addressed in the channel response, bank stability, and upland watershed technical areas. The results of the analysis performed in these areas will be used to determine the effects of the project on reservoir sedimentation.

During FY 1995, the historical land use in the Hickahala-Senatobia watershed will be evaluated along with the gauge data from Hickahala Creek. The GISSRM sediment yield model will be validated for Hickahala Creek. The major FY 1995 tasks are as follows:

a. Evaluate historical land use in Hickahala-Senatobia Creek watershed.

b. Evaluate gauge data from Hickahala Creek (sediment and water).

c. Validate the sediment yield model, GISSRM, using historical records from Hickahala Creek.

Streambank Stability

Channel bank stability depends on hydraulic parameters related to flow conditions and the characteristics of the materials in the banks. During FY 1995, an evaluation of bioengineering techniques (willow posts) and bendway weirs for bank stabilization, initiated in FY 1993, will be continued at the Harland Creek test reach. During FY 1994 five additional bioengineering bank stabilization techniques were designed (CSU contract). Those techniques will be installed in another portion of Harland Creek during FY 1995 and added to the monitoring program. The aerial visual inspection will be conducted for all 15 watersheds in cooperation with ARS. This inspection was not performed in FY 1994 due to contractor equipment problems. During FY 1995 past DEC design criteria relative to bank stabilization will be published. This will also include the observed performance of those designs. The major FY 1995 tasks are as follows:

a. Continue monitoring of Harland Creek bank stabilization test reach using bendway weirs and willow posts.

b. After installation, the five methods of bioengineering bank stabilization on the Harland Creek test reach will be monitored.

c. Conduct aerial visual inspection of 15 DEC watersheds for bank stability.

d. Publish past DEC design practices criteria for bank stabilization and performance of designs.

Design Tools

The design procedures and techniques used in the design of the different project features of the DEC project have the potential for national and international applications. To demonstrate their applicability, the procedures need to be organized into a systematic method, documented, and guidance prepared to illustrate their use.

Of particular interest is the method for designing the height, spacing, and sequence of construction of grade control structures. In FY 1995 the design tool that will be completed in the channel response work on Hotophia Creek sediment reduction study is an excellent example. The design tools proposed in this work will formalize those procedures, document their application, and train the Vicksburg District hydraulic engineers in their use. Although not listed in the design tools technical area, design tools discussed within the task areas of channel response, reservoir sedimentation, and bank stability are being developed from those efforts as presented. The major FY 1995 tasks are as follows:

a. Continue organization of procedures into a systematic method.

b. Validate the method on a DEC watershed.

c. Prepare guidance that will illustrate their application to a DEC and non-DEC watershed.

Technology Transfer

Technology transfer is an important part of the DEC program and will be given high priority at WES during the monitoring program. With the Vicksburg District approval, WES personnel will present results at national and international technical conferences and symposiums. When appropriate, WES will host workshops and training classes for both Corps and non-Corps personnel. WES will annually report on the DEC monitoring program using several different formats. For FY 1995, the following activities in technology transfer will be accomplished:

a. A detailed WES technical report on monitoring, data collection, data analysis, and project evaluation with emphasis on results, conclusions, and project performance.

b. An updated engineering database on the Intergraph system including aerial photos, surveys (channel and structural), and results of numerical studies to be provided to theVicksburg District.

c. Individual efforts that need to be documented in detail to be presented in WES technical reports or miscellaneous papers.

9 General Assessment After 3 Years

As stated at the beginning of this report, the objective of the FY 1994 report is to document the state of the DEC based on the WES monitoring activities. This chapter will summarize and give the general assessment of the DEC project after 3 years of WES monitoring. Thus the assessment presented herein is a summary of this report and the FY 1993 report (Raphelt et al. 1995). For clarity and ease of reference, the assessment is presented based on the major monitoring activities and associated DEC features.

Data Collection

The data collection effort has provided the following:

a. Water level and crest gauges have been installed on 19 DEC streams. These include pressure gauges, ultrasonic gauges, and crest gauges.

b. Failures of gauges have been limited and improvements have continued to be applied to the various gauge types to increase dependability.

c. Stream gauging for discharge data has been reasonably successful; however, on some streams only limited data have been collected. Alternative methods for obtaining discharge will be installed in FY 1995.

d. Stage-discharge curves and stage and discharge hydrographs have been developed for 16 of the DEC streams.

Engineering Database

The engineering database effort has provided the following:

a. The database is developed on an Intergraph 6040 workstation.

b. The database contains the following for 13 of the DEC watersheds (Abiaca, Batupan Bogue, Black-Fannegusha, Burney Branch, Cane-Mussacuna, Coldwater, Hickahala-Senatobia, Hotophia, Hurricane-Wolf, Long, Otoucalofa, Pelucia, and Toby Tubby):

(1) SCS curve numbers on a 1-acre grid, 1:24,000 digital quadrangle maps and DEM's, and streams and roads from the 1:100,000 USGS DLG's.

(2) All major tributaries and highways for the 15 DEC watersheds from the 1:100,000 digital DLG files.

(3) Spot-View satellite photography as a visual reference for all project features and satellite photography at 10-m resolution.

(4) Locations and design parameters for all the Vicksburg District existing structures for riser pipe, low- and high-drop structures; bank stabilization; and box-culvert grade control structures.

(5) Locations of proposed and constructed levees, floodwater-retarding structures, and channel improvement and box control structures.

(6) Land use data for Abiaca, Cane-Mussacuna, Coldwater, Hickahala-Senatobia, Hurricane-Wolf, and Long Creeks watersheds on a 1-acre grid.

(7) Soil type data for Abiaca, Batupan Bogue, Black-Fannegusha, Burney Branch, Cane-Mussacuna, Coldwater, Hickahala-Senatobia, Hotophia, Hurricane-Wolf, Long, Otoucalofa, Pelucia, and Toby Tubby Creek watersheds on a 1-acre grid.

(8) Elevation and slope data for Abiaca, Batupan Bogue, Black-Fannegusha, Burney Branch, Cane-Mussacuna, Coldwater, Hickahala-Senatobia, Hotophia, Hurricane-Wolf, Long, Otoucalofa, Pelucia, and Toby Tubby Creeks watersheds on a 30-m grid.

Channel Response

Observations of the channel response for the various monitored sites are as follows:

a. Abiaca Creek watershed, Site 3, indicated the following:

(1) The streambed is composed primarily of sand with minor amounts of gravel. The banks are generally well-vegetated with mature

vegetation down to the low-water surface. Generally the site is stable.

(2) The outside bank of the bendways is eroding.

(3) Notable bank erosion on the upstream portion of this site extends upstream of Highway 17 where a bend is actively migrating.

(4) Thalweg profiles vary on the order of 2 to 3 ft vertically.

(5) Channelization of the lower basin during the early 1920's set in motion a complex cycle of channel incision and widening.

(6) Continued mining of sand and gravel in the watershed complicates rehabilitation of the watershed.

b. Abiaca Creek watershed, Site 4, indicated the following:

(1) Site 4 is located approximately 1.8 miles downstream of a major sand and gravel processing operation that can be associated with an increased supply of suspended and bed material load.

(2) Streambanks are relatively stable and the bed gives the appearance of an aggraded reach.

(3) The most significant change in this watershed has occurred at the confluence of Abiaca and Coila Creeks, indicating aggradation.

(4) The island immediately downstream of the confluence has shifted to a bar attached to the right bank, with all flow along the left bank.

(5) Upstream of the confluence of Abiaca and Coila Creeks, Abiaca Creek is about 49 percent controlled.

c. Abiaca Creek watershed, Site 6, indicated the following:

(1) This is the site of the Pine Bluff gauging station with records from 1963 to 1980, which has been reactivated and includes a pumped sediment sampler.

(2) Thalweg profiles indicate that local variation has occurred on the order of 2 to 3 ft vertically. Generally the site is stable.

d. Abiaca Creek watershed, Site 21, indicated the following:

(1) The Vicksburg District has designed a sediment trap basin for this location by setting the levees back and allowing frequent overflow of the stream. The sediment trap, which is to be located immediately upstream of the wildlife area, has not been constructed.

(2) Thalweg profiles indicate that local variation has occurred on the order of 2 to 3 ft vertically, but the site is relatively stable.

e. Coila Creek, Site 5, indicated the following:

(1) The site has a watershed area very similar to Abiaca Creek Site 4, and allows the comparison of two almost equal size drainage basins.

(2) Coila Creek has a high proportion of the basin controlled by SCS reservoirs, and the gravel mines on Coila Creek are not as active as those along Abiaca Creek.

(3) The thalweg profile indicated that some aggradation has occurred on the downstream portion of this site with little or no changes in bed elevations for the remaining portion. Generally the site is considered stable.

(4) The downstream 500 ft of Coila Creek is actively eroding and the confluence with Abiaca Creek may shift downstream soon.

f. Burney Branch, Site 12, indicated the following:

(1) The two high-drop structures, constructed in 1982, have been very successful in rehabilitating this reach.

(2) Since 1984, several major channel stabilization projects have been constructed upstream.

(3) Surveys indicate that the upstream sediment yield was greater than planned.

(4) Thalweg profiles indicate that Burney Branch is relatively stable with only minor fluctuations in the bed profile.

(5) Only a minor portion of the lower portion of this reach is at risk of failure.

(6) This site is an excellent example of effective channel stabilization.

g. Fannegusha Creek, Site 2, indicated the following:

(1) This reach represents a very unstable, sand bed channel and is very active.

(2) A low-drop structure, constructed in 1993, appears to have caused aggradation upstream of the structure, and degradation has occurred immediately downstream of the structure. Most of the aggraded

reach is being colonized by grasses and young willows, except the left bank immediately upstream of the bridge, where kudzu growth is heavy.

(3) The bridge in the reach was replaced in 1994 due to channel widening. Bed degradation upstream of the bridge will continue as a headcut progresses upstream.

(4) The channel will continue to widen as oversteepened banks, due to previous bed degradation, continue to fail.

(5) The lower portion of the reach is in very resistant clay, and old, rotational failures are present, but these failures give no appearance of recent movement.

h. Harland Creek (upper site), Site 1, indicated the following:

(1) Harland Creek is a mixed sand and gravel bed stream with a meandering planform.

(2) The stream is unstable, with bank erosion and significant channel widening.

(3) Several areas of massive bank failures have been identified, and these failure sites, along with bed and bank erosion, provide a high sediment yield to the downstream.

(4) Bank erosion is due to local hydraulic forces, and channel degradation should not be occurring unless the upstream sediment supply is significantly reduced.

(5) The thalweg profile has fluctuated somewhat; however, no significant changes in a system context are evident.

i. Harland Creek (lower site-willow post), Site 23, assessment is indicated below. In the "Bank Stability" section, additional discussions relative to bendway weirs and willow posts are presented.

(1) Aggradation with large gravel bars in unusual locations for a meandering stream indicate that the deposits occurred during the recession of a major flood event. It is expected that lower flows will continue to rework these deposits.

(2) One reason for the aggradation may be an oversupply of coarser sediment to the stabilized sinuous reaches in which the tight bends may be causing hydraulic energy loss.

(3) The reach slope appears to be approaching stability, and bank erosion is primarily due to local hydraulic forces.

(4) Historic planform patterns in this reach indicate frequent cutoffs that are now prevented by willow post and bendway weir constructed features.

(5) Willow post mortality has been high, and an overall survival rate of 42 percent was determined in the fall of 1994, down from an 80 percent survival in the spring of 1994.

(6) Survival rate improved for posts used in conjunction with 1 ton-per-linear-foot riprap toe; however, mortality rate was very high landward of the riprap toe if the landward fill did not drain adequately.

(7) Scalloping between bendway weirs has begun to be healed by colonizing vegetation, and filling between the riverward tips of the bendway weirs was observed.

j. Hickahala Creek at the confluence of South Fork Hickahala Creek, Site 11, indicated the following:

(1) Hickahala Creek is a major tributary to the Coldwater River. The study reach includes two existing structures and was selected to monitor the response of the structures.

(2) The upstream drop structure appears to have changed little, although some rock displacement upstream has occurred. The South Fork drop structure is a newer, grouted rock structure. Minor cracking of the grout was observed.

(3) The structures have given this site an opportunity to stabilize, and upstream of the South Fork and the upstream Hickahala Creek structure have resulted in significant stability. A box culvert for the downstream bridge will help in stabilizing the downstream reach; however, the site downstream of the two upstream structures is generally unstable and adjustment will continue.

(4) The upstream portion of the study reach is relatively stable with a sand bed, and the lower portion is actively incising into a cohesive clay bed.

(5) The confluence of South Fork with Hickahala Creek is eroding badly, with high flows from Hickahala Creek now entering the tributary at a location about 150 ft upstream of the previous confluence. The confluence also has significant debris from fallen trees.

(6) Beaver dams are present, and the water ponded upstream of the dams has caused the banks to fail.

k. Hotophia Creek and Marcum Creek, Site 13, indicated the following:

(1) The study reach includes the confluences of Marcum Creek and Deer Creek with Hotophia Creek.

(2) A low-drop is located at the downstream extent of Hotophia Creek and a high-drop is located on Hotophia Creek immediately downstream of the confluence with Marcum Creek. Two low-drops are situated on Deer Creek, and one low-drop is located on Marcum Creek approximately 800 ft upstream of the confluence with Hotophia Creek. Two additional high-drops, one within the reach and one upstream of the reach, were constructed in 1994.

(3) This site is important because of the complexity of the various constructed elements and the need to document channel response to the high-drop grade control.

(4) The primary change in the thalweg profile of Hotophia Creek is the result of construction of the downstream high-drop structure and the filling of the next upstream structure.

(5) The downstream extent of the study reach is the older low-drop structure, which is downstream of Highway 315. This structure appears unchanged with a significant drop at the downstream extent of the riprap. However, the water depth upstream seems to have increased and debris is present on the channel bed.

(6) A severe gully is present on the left bank immediately downstream of the Highway 315 bridge and adjacent to an existing drop pipe. Another gully within the riprap is apparently causing sediment accumulation upstream of the high-drop structure.

(7) Marcum Creek has continued to degrade upstream of the Hotophia Creek high-drop influence.

(8) The channels are expected to continue to adjust toward equilibrium with the control imposed by the drop structures.

l. James Wolf Creek, Site 19, indicated the following:

(1) The stream has a sand bed, and at low-flow conditions, the channel may be dry.

(2) A low-drop structure appears to be stabilizing the bed of the stream; however, the banks remain unstable due to the significant depth. The drop structure has required significant repair since construction

and is presently in need of significant repair. Two additional drop structures are present downstream of the monitoring reach.

(3) The thalweg profile indicates that no significant change has occurred during the past 3 years.

(4) A large beaver dam exists on the upstream riprap approach. A backwater condition exists from the beaver dam to the upstream tributary on the left bank.

(5) Downstream from the upstream bridge, willows are establishing on an island in the center of the channel, and heavy kudzu growth dominates the bank vegetation.

m. James Wolf Creek, Site 19, indicated the following:

(1) This reach provides an excellent opportunity to document a stable, channelized, sand-bed stream.

(2) The channel is relatively stable and is transporting minor amounts of gravel in a sand bed. Upstream of the bridge, the channel exhibits some meandering. Downstream of the bridge, the channel is stable.

(3) The remnants of spoil piles indicate that downstream of the bridge, the channel has been channelized.

(4) Bridge abutments at the road crossing in the center of the study site are in poor condition.

(5) The downstream banks are relatively stable and conveyance is good, which is in direct contrast to the upstream portion of the study site. The islands creating divided flow and collecting debris cause reduction of conveyance.

(6) Based on the channel profiles, channel degradation has continued to take place with vertical changes in the range of 2 to 4 ft.

n. Lick Creek, Site 8, indicated the following:

(1) The upstream extent of the site is incising into resistant clay.

(2) Riprap placed at the bridge as a temporary measure during construction of the high-drop structure has slowed the incision upstream and downstream of the bridge.

(3) Degradation is continuing downstream of the structure and can be expected to continue after the closure of the structure.

(4) The high-drop structure will improve the stability of the upstream channel reach. Backwater from the structure should assist in halting the upstream incision if the knick zones have not progressed too far upstream to be affected by the high-drop.

(5) Left bank drainage upstream of the bridge is poor, with standing water in the adjacent field. Channel incision and a saturated left bank may combine to result in greater instability than in other similar streams.

o. Long Creek, Site 20, indicated the following:

(1) Five low-drop structures and a weir are present in this reach.

(2) This site will furnish unique information pertaining to channel adjustment in a channel that is limited in width adjustment.

(3) Portions of the reach are very unstable and are presently incising. The reach downstream of the existing structures has a clay bed that is slowly incising.

(4) Aggradation is taking place on the upstream portion, and downstream of the weir a headcut is moving into the structure. Headcutting is also present in the downstream portion of the site.

(5) A significant number of beaver dams are present at the site.

p. Nolehoe Creek, Site 7, indicated the following:

(1) The reach is representative of the suburban development occurring in the metro-Memphis area and has degraded by approximately 4 ft.

(2) A major cutoff of the channel had been made in the last 10 years.

(3) The channel is extremely unstable and is deeply incised. Bed material load ranges in size from fine sand to gravel.

(4) The changing land use suggests that discharges are increasing and will supply less sediment in the future. In the absence of grade control, floodwater detention should be considered.

q. Otoucalofa Creek, Site 14, indicated the following:

(1) This site provides a unique opportunity to observe riprap subjected to severe degradation.

(2) The reach is actively incising, and this incision is occurring at an elevation below the recently placed stone.

(3) A steep knick zone exists just beyond the upstream extent of the study reach.

(4) Downstream of the Mt. Liberty Church road bridge the channel is relatively wide and meandering with some point bar formation. The banks have been revetted and only minor launching of the revetment has been observed. Upstream of the bridge, longitudinal toe riprap and dikes have been placed on what was the hard clay bed of the channel. Incision has progressed up through the prior bed and formed a narrow inner channel that is steep and active and generally below the riprap.

(5) At the downstream extent of the reach, the left bank is protected by a series of dikes that are experiencing severe launching; however, the dikes remain functional.

r. Perry Creek, Site 16, indicated the following:

(1) The site is unique because within the study reach, the channel moves from a deeply incised stream at the downstream end to a stream that might have existed prior to channelization at the upstream end.

(2) Four low-drop structures were completed during 1994, and this site will allow the investigation of the effects of four structures in series. The structures are performing well and the system seems to be stabilized.

(3) The grade control structures have significantly reduced downstream sediment load by causing aggradation, reducing the probability of continued headcutting, and by improving bank stability.

(4) Vegetation density is increasing along the study reach and beavers have constructed numerous dams.

(5) Gullying existed within the construction area and downstream of the left bank riprap at the third structure entering the channel in the toe riprap.

(6) Another gully, which should be considered for a drop pipe, is present on the right bank downstream of the left bank existing pond and drop pipe.

(7) The bed of the channel is very steep upstream of the third structure and composed of fractured ironstone and clay. Upstream of the fourth structure and within the construction clearing on the left bank is a severe gully.

s. Red Banks Creek, Site 9, indicated the following:

(1) This is the only DEC monitoring site using chevron dikes and longitudinal dikes for channel stabilization.

(2) The bed sediment load is sand, and the stream flows in a deeply incised and widened, straight channel.

(3) Degradation in the range of 1 to 3 ft has occurred downstream of the chevron weirs, and the pool downstream of the most downstream chevron weir is in excess of 10 ft in depth.

(4) The chevrons were observed to be displaced and degradation has occurred. Less stone has been displaced in the second weir than in the first, and the two upper weirs exhibit significant displacement.

(5) The longitudinal riprap has remained stable with launching only in the reach downstream of the weirs.

(6) The combination of longitudinal riprap and upstream aggradation resulted in a wider channel upstream than downstream of the weirs as the channel aggraded upstream and degraded downstream.

(7) Shell Oil pipeline interests constructed a drop structure downstream of the study reach. Backwater was observed up to the downstream extent of the study reach.

t. Sarter Creek, Site 15, indicated the following:

(1) The channel is relatively small. This may provide the opportunity to test low-cost grade control structures, such as gabion structures, to stabilize the channel.

(2) The site is unusual in that it has remained relatively unchanged since channelization; however, it is apparent that headcutting is taking place and the bed is degrading.

(3) The bank stability remains good because the bank height is low, but the degradation potential is high.

(4) Beaver dams control most of the upper portion of the reach and no headcuts are present.

u. Sykes Creek, Site 17, indicated the following:

(1) Thalweg profile indicates that little change has occurred at the site.

(2) There are several factors relative to the site, such as berm formation, depth of sand in the bed, and thalweg comparison, which indicate that the channel could be considered in quasi-equilibrium.

(3) Upstream of the county road bridge a large, tight bend is active and in the process of making a cutoff. The alignment to the downstream bridge is presently relatively straight, and the new alignment is uncertain.

v. East Worsham Creek, Site 18a, indicated the following:

(1) The Worsham streams are deeply incised and active with portions of the channel composed primarily of very erosion resistant clay and numerous outcrops of ironstone and clay, which provide a degree of vertical stability.

(2) Ten low-drop structures have been constructed in this study reach, and the grade control has raised the channel bed to reduce the bank instability.

(3) The thalweg profile indicates that there is a general tendency for this site to degrade.

(4) Sediment transport is significantly controlled at lower discharges by beaver dams.

w. Middle Worsham Creek, Site 18b, indicated the following:

(1) Channel widths are narrow through the site.

(2) The thalweg profile indicates that degradation has occurred in the downstream portion of the site. The bank stability has decreased as the channel has degraded.

(3) Immediately upstream of the confluence of West and Middle Worsham, the channel has a shallow depth of sand, less than 2 ft. Upstream of this confluence, the channel is narrower and knick zones are present.

(4) Near the upstream structure, knick zones are present and massive bank failures are present. Upstream of the third structure, knick points are present at several locations. Also, sediment has filled to the crest of this structure.

x. West Worsham Creek, Site 18c, indicated the following:

(1) Three ARS-type low-drop structures are present at this site.

(2) Thalweg profiles indicate that some degradation has occurred in the downstream portion of the site and that aggradation has occurred immediately upstream of the newer, second structure.

(3) Filling has occurred to the weir crest of the second structure, while the older, first structure has not filled. This indicates that the improved hydraulic control of the newer design results in improved performance.

(4) Beaver dams are abundant immediately downstream of the first structure and upstream of the second structure.

(5) The gully into the second structure has been rehabilitated.

Hydrology

The hydrology effort conducted under the DEC Monitoring Program has provided evaluation and modifications to hydrologic models that can be used in the future for accurate runoff computations. The use of these types of models is important to the overall DEC project due to the large number of ungauged watersheds and the need to produce adequate results with a limited amount of data. These models will be used to provide better designs and evaluation of existing channels.

The hydrology effort has provided the following:

a. It is recommended that the CASC2D model be used as an aid in the design and evaluation of streambank erosion and grade control structures in the future, because fewer subbasin stream gauge data need to be collected.

b. Compared to a HEC-1 model, the CASC2D model produced more realistic results in terms of hydrograph shape and volume of runoff and offers more flexibility when performing sediment studies.

c. Where sufficient subbasin stream gauge data are available for calibration purposes, models such as HEC-1 can reproduce the observed hydrograph reasonably well.

d. If accurate spatial data and subbasin stream gauge data are lacking, then a CASC2D model or a HEC-1 model may produce questionable results.

e. Modifications were made to the CASC2D model to eliminate numerical instability, which occurred during low base flow start-up conditions.

f. CASC2D models of Hotophia Creek, Hickahala-Senatobia Creek, Goodwin Creek, and Batupan Bogue Creek watersheds have been constructed. Preliminary runs of the models of Hotophia Creek, Hickahala-Senatobia Creek, and Goodwin Creek watersheds have been accomplished. The Batupan Bogue Creek watershed model needs the initial channel conditions and the final elevation grid to be added.

g. It is recommended that the channel routing component of the CASC2D model be revised as soon as possible to represent the channel cross sections more realistically to improve the timing of the simulated runoff hydrographs. It is also recommended that the channel routing component be uncoupled or separated from the overbank routing component for modeling overbank flows.

h. It is recommended that the CASC2D model be enhanced by adding sediment yield and transport subroutines for both the overland flow and channel routing components.

i. The GISSRM model is very similar to the HEC-1 model using SCS unit hydrograph procedures and Muskingum-Cunge channel routing techniques.

j. The GISSRM model's synthetic routing methodology for flow and sediment is giving reasonable results when compared to observed data for the historical storm runs as well as the annual yield prediction.

k. GISSRM models of other DEC watersheds will be constructed in future years.

Hydraulic Structures

The evaluation of hydraulic structures has provided specific information on the relative condition and associated problems of individual drop structures. In addition to the structures previously presented within the monitored channel response sites, the condition and problems (if any) of the drop structures are provided in Table 35. Since the DEC hydraulic structures were not inspected in FY 1994, the assessment presented is based on the FY 1993 report (Raphelt et al. 1995).

Bank Stability

The evaluation of bank stabilization works has provided specific information on the relative condition and effectiveness of various types of stabilization used on the DEC project. The installation of the innovative design of bendway

weirs and bioengineering design of willow posts on Harland Creek provides an additional methodology that has been significantly monitored in detail from inception to the present.

The evaluation of bank stabilization methods on the DEC Project has provided the following:

a. Goodwin Creek:

(1) Longitudinal and transverse dikes were inspected and evaluated for this site.

(2) Goodwin Creek is a mixed-bed stream with significant portions of the bed material comprised of gravel.

(3) Bank erosion in the narrow reaches was concentrated at sites where alternate bars or slight channel curvatures forced the flow to impinge directly onto the bank toe.

(4) Use of intermittent stone dikes in the relatively sharp bend of the study reach has been successful. Scalloping between hardpoints was minimal.

(5) The longitudinal peaked stone toe protection was functioning without disturbance from the stream, and apparently no channel incision had occurred since these features were constructed. The protection is covered with kudzu.

b. Little Bogue:

(1) Riprap along both banks for approximately 1 mile upstream and downstream of the ARS-type low-drop grade control structure and the confluence with Caffe Branch was inspected and evaluated for this site.

(2) The reach included an outcrop of shale in the channel bed that acts as a grade control in the channel.

(3) The channel bed is primarily sand contributed from eroding side channels and gullies.

(4) The toe riprap in the lower reach is very unstable for walking, and frequently launches due to this disturbance. At several locations the streambed has been covered with riprap, either as a result of bank riprap launching, a construction access road, or as an intended grade control. Within this reach the right bank is protected using a stone toe and the left bank by a hardpoint. A scour hole results and a very irregular bank results downstream.

(5) Rock size for the lower reach appears to be too small, based on the amount of launching and transport noted; however, the rock may have provided a stable substrate long enough to allow vegetation to become the dominant bank stabilization.

(6) The middle reach will remain stable unless the shale outcrop fails, which would undermine the existing bank stabilization and threaten the grade control structure.

(7) The channel takes a more stable appearance upstream, with some berming along the toe of the bank. In this reach the riprap is resting on the upper surface of the shale outcrop, and the shale has been eroded approximately 3 ft vertically. Launched riprap does not remain on the shale surface and is transported downstream.

(8) The upper reach remains relatively stable, although cutoffs of the tight meanders may continue.

c. Otoucalofa Creek:

(1) Riprap longitudinal and transverse dikes extend upstream and downstream of the Mt. Liberty Church road bridge.

(2) Approximately 3 miles downstream of the study reach, the creek has been channelized for 10 miles to the confluence with the Yocona River. The channel is actively incising within the study reach probably as a result of the channelization downstream.

(3) Sand and gravel are transported as bed load, and the bank material is composed primarily of sand with sufficient amounts of silt and clay material such that steep banks remain stable.

(4) The site has six spur dikes along the left bank of the channel downstream of the bridge and five spur dikes upstream of the bridge with all dikes spaced approximately 100 ft apart.

(5) A series of 17 spur dikes were constructed along the left bank and an equal number of dikes were constructed on the right bank. In addition, toe riprap was used to help stabilize the banks.

(6) The two downstream left bank dikes are experiencing severe scalloping; however, they are remaining functional.

(7) The toe riprap and dikes are stable and have not been displaced by the flow in the stream. This indicates that the rock was properly sized and constructed. Sediment has deposited behind much of the toe riprap and between some of the dikes.

(8) The two channel stabilization projects provide lateral control of the channel; however, no grade control for the channel was included. Consequently, the bed elevation of the channel is continuing to decrease as the channel further incises.

(9) In many locations, the toe riprap placed along the banks of the channel is elevated above the present channel invert. A vertical drop of approximately 1 ft from the bottom of the toe riprap to the streambed was observed in many locations. At some locations, the riprap is launching into the stream. The spur dikes are also vulnerable to continued incision of the channel with the low-flow channel incised below the bottom of the dike. If the stream continues to incise, the integrity and stability of the toe riprap and dikes will be compromised.

d. Red Banks Creek:

(1) Riprap longitudinal and transverse dikes extended for a distance of approximately 1 mile upstream of the Watson-to-Moscow highway in conjunction with v-notch weirs constructed of riprap.

(2) The channel is relatively straight and has generally uniform characteristics along the study reach, which is a result of channelization of the creek. The stream is oversteepened and actively incising.

(3) Significant sand and gravel are transported through the reach.

(4) The bank material is composed primarily of sand with a sufficient amount of silt and clay that the very steep banks remain relatively stable.

(5) A total of 12 segments of toe riprap were placed along the right bank of the channel with the toe riprap tied into the banks of the channel through a series of 28 tiebacks. The riprap was placed at the rate of 1 ton per linear foot.

(6) Five v-notch weirs were constructed of riprap placed across the channel in the study reach. Each weir provides a 2-ft rise on the bed of the channel, had a typical length of 14 ft, and was designed to serve as a grade control structure.

(7) The toe riprap and spur dikes have been used successfully to stabilize the banks of the channel. The increase in bank roughness does not appear as though it will significantly change the future aggradation or degradation of the channel in comparison with degradation that would have occurred without the riprap.

Bank stabilization on Harland Creek is an experimental application of two significantly different approaches: bendway weirs and willow posts. The use of bendway weirs as bank stabilization on Harland Creek is the first such application of that technology within the DEC. Willow post technology has been expanded in recent years by the Illinois State Water Survey and use of this technology within the DEC seemed like a reasonable alternative for consideration. Since these two applications are significantly different, the results of the evaluations and monitoring by CSU and WES for bendway weirs and willow posts on Harland Creek are presented separately in the following paragraphs.

Harland Creek is a tributary to Black Creek with an average width of approximately 95 ft and average depth of approximately 6 ft. The channel has an actively meandering, sinuous planform with an average annual bank migration rate of approximately 14 ft per year. Significant amounts of sand and gravel are transported through the reach.

a. Harland Creek application of willow posts indicated the following:

(1) In February 1994, 9,383 dormant willow posts were planted. Virtually all of the posts sprouted in the spring, but by June 1994, the survival rate of the willow posts was measured at 80 percent and by October 1994, 42 percent.

(2) The survivability increases as the diameter increases from 1 to 7 in., reaches a minimum when the elevation of the post at the ground line is 2 ft below or 7.5 ft above the low-water surface elevation. The aspect of the streambank data indicates that banks facing east or west have the greatest survival rate and banks facing north are better than banks facing south.

(3) Relative to survivability, the effect of canopy was inconclusive, indicating that the amount of canopy may not be a primary factor within the limits of the data measured for the Harland Creek site.

(4) The high mortality of willow posts can be attributed to insect infestation, lack of aerobic rooting conditions caused by impermeable soils, damage from high velocity, sediment-laden flood flows abrading the new growth, improper backfilling during construction, lack of sufficient moisture, and constant anaerobic rooting conditions due to standing water.

(5) The Harland Creek data suggest that the high mortality of the lower row of posts was the result of streamflow damage and the anaerobic conditions in standing water or poor soils. The high mortality of the upper rows, more than 6 ft above low water, may represent a site-specific threshold of sufficient moisture for sustaining growth.

(6) Beavers did chew on the installed willow posts and the 1/2- to 3/4-in.-diameter young branches. While the beaver activity has been fairly intense in at least two reaches, project stability has not been compromised.

(7) Many of the posts, alive or dead, contributed to the stability of the bank by increasing the bank strength and by holding failed soil blocks.

(8) Results with willows planted landward of longitudinal peaked stone toe protection were mixed. If the willows were planted on a sloped bank at an elevation greater than the crest of the stone protection, overall survival and growth rates were good. If the willows were planted in the depositional area landward of the stone protection at an elevation lower than the crest of the protection, water puddled and the willows died.

(9) Other woody and nonwoody species were observed to be colonizing within the willow post sites. Because of the increased hydraulic roughness within the sites, sediment was observed to be depositing and was providing suitable conditions for riparian invasive species.

(10) While survival rate is certainly one measure of project success, long-term stability will depend on the initial (post planting) stability of the bank and the ability of native plants to colonize the bank.

(11) Except for one reach, no repair of willow post stabilization has been required with R_c/W values of from 1.03 to 1.50. Although preliminary, it appears that the willow post stabilization should function in the average range of values of R_c/W, i.e., 2.0 to 2.3.

(12) Future installations of willow posts for bank stabilization can be enhanced by assuring that the willow post holes are adequately backfilled and tamped after post installation; preventing erosion of the toe of the bank during the establishment period; and interplanting of the willow posts with other species. The use of branch matting, geotextile, or fascines or similar materials buried along the rows would be of benefit in enhancing aggradation within the post system, which can provide rooting medium in areas of poor soil conditions.

b. Harland Creek application of bendway weirs indicated the following:

(1) Of the 54 weirs constructed, 35 were initially located and angled incorrectly. Since construction, 5 of the weirs have been relocated, 11 weirs were reangled, and 19 weirs have not been modified and are still incorrectly located or angled to some degree.

(2) Since the weirs were constructed, the site has been subjected to at least four out-of-bank flows in the first year. In spite of these conditions and events, the weirs have generally performed very well in stabilizing the outer bank of the bends. Most banks are rapidly maturing as a mixture of plants (grasses and weeds) and volunteer willows have invaded these areas.

(3) Excessive bank scour and scalloping have occurred between the last two weirs and downstream of the last weir in each bend with weirs. In the six reaches with weirs the scour between the last two weirs has naturally healed and stabilized. The bank instability downstream of the last weir has naturally stabilized in four of the six bends. One of the two problem areas was caused by contractor error where one weir was left out and the immediate upstream and downstream weirs were incorrectly angled between 35 and 45 degrees upstream.

(4) Very little rock settling has been observed. A few weirs appear to have been damaged by high flows and/or debris where the crest elevation near the bank end is lower than the crest elevation at the stream end. Weir performance does not appear to have been compromised and the bank is stable in this area.

(5) Scour holes at the stream ends of the weirs have appeared stable, but did not deepen enough to launch rock from the end of the weir. Bar formations were observed to be forming and connecting between adjacent weirs.

(6) Weir angle is extremely important. Weirs incorrectly angled near the center of the weir field do not appear to compromise performance. Weirs incorrectly angled near the downstream end of the bend or in an area of high flow concentration can lead to bank erosion problems.

(7) Bendway weir length, spacing, and angle with the flow are critical factors that contribute to the success or failure of the system.

(8) For the most part bendway weirs have functioned adequately with R_c/W values of from 1.89 to 2.77.

(9) One of the stream features identified as being needed for successful aquatic habitat improvement or restoration was the presence of woody debris. Woody debris correctly positioned within a weir field will stay in place even with repeated out-of-bank flows. The woody debris was either in place or moved into place and has stayed there.

(10) Future installations of bendway weirs for bank stabilization can be enhanced by assuring that the particular flow condition causes the flow to be angled into the weirs where the weirs cannot redirect flow and actually act as a flow divider. Possible repairs to this situation would be (a) installation of longitudinal peaked stone toe protection between the last two weirs and continuing 100 ft downstream from the root of the last weir, (b) ensuring that the stone in the weir key is flush with the surrounding bank, and (c) allowing the point bar to reshape after project installation, then build one or two structures on the point bar that would correctly aim high-flow currents toward the weir field at the downstream end of the bend.

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Table 1
DEC Stage and Discharge Available

Site Name	WES Site	Stage (DSS) for FY			Rating Curve[1]	Discharge (DSS) for FY		
		92	93	94		92	93	94
Harland	030130 030120 030140		X		US-95		X	X
Fannegusha	030220 030210		X X	X X				
Abiaca	010320	X	X	X	US-95	X	X	X
Coila	010520	X	X	X	US-95	X	X	X
Nolehoe	050760 050730 057400	X X	X X	X X	US-95	X	X	X
Lick	050820		X	X	US-95	X	X	X
Red Banks	050910		X	X				
Lee	051020		X	X	US-95		X	X
Hickahala	061120 061130	X X	X X	X X	TH-94	X	X	X
Burney Branch	041220 041230	X X	X X	X X	TH-94	X	X	X
Hotophia	071330 071340	X	X	X	TH-94	X	X	X
Otoucalofa	091420		X	X	US-95		X	X
Sarter	091520	X	X	X	US-95		X	X
Sykes	021720		X	X				
West Fork Worsham	021811	X	X	X	TH-94	X	X	X
Middle Fork Worsham	021821 021823	X X	X X	X X	TH-94	X	X	X
East Fork Worsham	021831 021833	X X	X X	X X	TH-94	X	X	X
James Wolf	061920 061930	X X	X X	X X	TH-94	X	X	X
Long	082020 082030 082050	X X X	X X X	X X X	TH-94	X	X	X

[1]US = USGS measurement.
TH = Curve developed using theoretical computations.
Last two digits indicate the fiscal year the curve was developed.

Table 2
Information Available on Engineering Database

Watershed	EI[1]	Soil Type[2]	Land Use[2]	Slope[1]	Curve Number[2]	Hydraulic Structures	Quad Maps	Spot Photo[3]
Abiaca	X	X	X	X	X	X	X	X
Batupan Bogue	X	X		X	X	X	X	X
Black-Fannegusha	X	X		X	X	X	X	X
Burney Branch	X	X		X	X	X	X	X
Cane-Mussacuna	X	X	X	X	X	X	X	X
Coldwater	X	X	X	X	X	X	X	X
Hickahala-Senatobia	X	X	X	X	X	X	X	X
Hotophia	X	X		X	X	X	X	X
Hurricane-Wolf	X	X	X	X	X	X	X	X
Long	X	X	X	X	X	X	X	X
Otoucalofa	X	X		X	X	X	X	X
Pelucia	X	X		X	X	X	X	X
Toby Tubby	X	X		X	X	X	X	X

[1]Grid size = 30 m X 30 m
[2]Grid size = 1 acre
[3]Satelite photograph at 10-m resolution

Table 3
Summary Table for All Streams at 2-Year Event

Site	Stream	Year First Surveyed	Segment	At Lesser of 2-year Flow or Bankfull				D_{50}, mm
				Discharge cfs	Width, ft	Depth, ft	Slope	
1	Harland	Jan 1992	1	3739	97.8	7.3	0.001 08	0.50
2	Fannegusha	Jan 1992	1	3325	74.1	8.1	0.001 07	0.31
			2		86.7	8.1	0.000 78	
3	Abiaca 3	Jan 1992	1	3339	74.1	8.3	0.000 97	0.26
4	Abiaca 4	Jan 1992	1	3780	77.0	8.2	0.001 19	0.48
5	Coila	Jan 1992	1	4780	80.7	8.2	0.001 78	
6	Abiaca 6	Jan 1992	1	7095	125.0	11.6	0.000 50	0.50
7	Nolehoe	Jan 1992	1	978	37.7	4.5	0.002 53	0.62
			2		31.7	4.2	0.004 60	
8	Lick	Jan 1992	1	1581	55.8	4.7	0.002 66	0.62
			2		43.2	5.6	0.002 32	
9	Red Banks	Jan 1992	1	3951	113.8	6.2	0.001 52	0.51
			2		109.7	6.3	0.001 56	
10	Lee	Jan 1992	1	1377	50.1	5.5	0.001 39	0.39
11	Hickahala	Jan 1992	1	2158	58.7	6.7	0.001 32	0.51
			2		51.2	7.0	0.001 49	
			3		42.7	6.2	0.003 21	
12	Burney Branch	Jan 1992	1	2662	114.7	9.5	0.000 17	0.37
			2		97.3	6.6	0.000 78	
13	Hotophia	Jan 1992	1	3386	73.1	6.4	0.002 44	0.31

(Sheet 1 of 3)

Table 3 (Continued)

Site	Stream	Year First Surveyed	Segment	At Lesser of 2-Year Flow or Bankfull				D_{50}, mm
				Discharge cfs	Width, ft	Depth, ft	Slope	
13	Hotophia	Jan 1992	2		81.7	8.0	0.000 93	
14	Otoucalofa	Jan 1992	1	4617	89.9	9.0	0.000 98	0.39
15	Sarter	Jan 1992	1	1391	35	5.8	0.002 58	0.38
			2		54	5.7	0.001 12	
16	Perry	Jan 1992	1	1790	105.3	10.6	0.000 06	0.33
			2		91.6	6.9	0.000 34	
			3		50.7	5.7	0.002 04	
			4		48.7	4.7	0.004 21	
17	Sykes	Jan 1992	1	2542	78.3	5.7	0.001 73	0.34
18a	Worsham	Jan 1992	1	1935	48.4	6.4	0.001 82	0.26
			2		52.0	5.6	0.002 51	
			3		54.4	5.8	0.001 97	
			4		52.3	5.8	0.002 18	
18b	West Fork Worsham	Jan 1992	1	1096	45.3	5.2	0.001 49	0.32
			2		41.9	4.9	0.002 08	
			3		50.8	5.1	0.001 22	
			4		37.0	4.6	0.003 33	
18c	Middle Fork Worsham	Jan 1992	1	1153	34.2	5.6	0.001 82	0.29
			2		38.9	4.6	0.001 82	

Table 3 (Concluded)

Site	Stream	Year First Surveyed	Segment	At Lesser of 2-year Flow or Bankfull				D_{50}, mm
				Discharge cfs	Width, ft	Depth, ft	Slope	
18c (cont)			3		51.2	4.5	0.001 75	
			4		37.1	4.7	0.002 78	
19	James Wolf	Jan 1992	1	2189	78.3	5.9	0.001 18	0.36
			2		76.9	5.3	0.001 71	
20	Long	Jan 1992	1	2209	59.3	4.4	0.001 04	0.38
			2		68.7	3.5	0.001 70	
			3		47.3	3.8	0.002 60	
			4		60.6	3.3	0.002 68	
21	Abiaca 21	Jan 1993	1	5750	91.5	6.8	0.000 01	0.35
22	Hickahala	Jan 1993	1	n/a	n/a	n/a	n/a	n/a
23	Harland	Jun 1993	1	750	77.2	3.2	0.001 08	n/a

(Sheet 3 of 3)

Table 4
Abiaca Creek, Site 3, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 121 Cohesion, lbf/ft^2: 331

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	14.7	0	14.7	0	14.7			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	8	0	6	0	6	0	6	5	6

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 90 Minimum Slope: 0.000665

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	62.2	147.7	65.5	141.5	70	160.9

Table 5
Abiaca Creek, Site 4, SAM and BURBANK Analysis Summary

Unit Weight lb/ft^3: 121 Cohesion, lbf/ft^2: 331

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	14.7	0	14.7	0	14.7			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	0	0	6	0	0	0	0	0	0

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 56 Minimum Slope: 0 .001396

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	125	135.2	132.1	97.1	123.2	88.8

Table 6
Coila Creek, Site 5, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 121 Cohesion, lbf/ft^2: 331

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	14.7	0	14.7	0	14.7			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	0	0	0	0	0	0	0	0	0

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 25 Minimum Slope: 0.001941

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	232	93.7	260	88.6	272	91.7

Table 7
Abiaca Creek, Site 6, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 121 Cohesion, lbf/ft^2: 331

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	14.7	0	14.7	0	14.7			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	20	0	6	0	0	0	0	6	12

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 71 Minimum Slope: 0 .000623

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	112.7	195.8	146.5	99.2	145.1	85.1

Table 8
Abiaca Creek, Site 21, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 121 Cohesion, lbf/ft^2: 331

Segment	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	14.7	0	14.7	0	14.7			
	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	NA	NA	6	6	0	0	0	0	0

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 151 Minimum Slope: 0.000184

Segment	1992		1993		1994	
	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	NA	NA	80.8	227.2	60.9	5.4

Table 9
Burney Branch, Site 12, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 120 Cohesion, lbf/ft^2: 274

Segment	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	18.5	0	18.5	0	18.5			
	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	9 0	0 0	12 0	0 0	10 0	0 0	15 0	25 0	25 3

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 123, 96 Minimum Slope: 0.000158, 0.000762

Segment	1992		1993		1994	
	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	84.7 109.4	307.6 68.9	76.4 102.1	677.2 84.4	93.5 101	107.6 102

Table 10
Fannegusha Creek, Site 2, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 122 Cohesion, lbf/ft^2: 413

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	13.3	0	13.3	0	13.3			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	0	0	0	0	0	0	0	0	0

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 59 Minimum Slope: 0.000562

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	113.5	274	125.4	254.4	144.1	145.9

Table 11
Harland Creek, Site 1, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 122 Cohesion, lbf/ft^2: 413

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	13.3	0	13.3	0	13.3			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	0	0	0	0	0	0	0	0	0

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 17 Minimum Slope: 0 .00123

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	470.6	81.1	500	65.6	505.9	81.3

Table 12
Harland Creek, Site 23, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 122   Cohesion, lbf/ft^2: 413

Segment	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	13.3	0	13.3	0	13.3			
	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	NA	NA	NA	NA	2	2	2	2	2

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 68   Minimum Slope: 0.00093

Segment	1992		1993		1994	
	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	NA	NA	NA	NA	113.2	116.4

Table 13
Hickahala Creek, Site 11, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 125   Cohesion, lbf/ft^2: 450

Segment	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	8	0	8	0	8			
	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	0	0	0	0	0	0	0	17	17
2 of 4	0	0	0	0	0	0	0	0	0
3 of 4	NA	NA	0	0	0	0	0	0	0
4 of 4									

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 27, 27, 58   Minimum Slope: 0.0011, 0.00103, 0.00103

Segment	1992		1993		1994	
	Width	Slope	Width	Slope	Width	Slope
1 of 4	200	100.9	200	94.3	181.5	134.1
2 of 4	218.5	118.8	151.8	192.2	166.7	137.1
3 of 4	NA	NA	65.5	291.6	65.5	285.7
4 of 4						

Table 14
Hotophia and Marcum Creeks, Site 13, SAM and BURBANK Analysis Summary

Hotophia Creek

Unit Weight, lb/ft^3: 120 Cohesion, lbf/ft^2: 274

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	18.5	0	18.5	0	18.5			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	NA	NA	NA	NA	0	0	0	0	0
2 of 4	78	1	22	0	45	0	55	59	67
3 of 4	93	0	15	0	47	2	47	50	53
4 of 4									

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 79, 93, 53 Minimum Slope: 0.00159, 0.00062, 0.0080

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4	NA	NA	NA	NA	92.4	153.1
2 of 4	78.5	297.4	69.9	293.2	88.2	151.2
3 of 4	88.7	585.2	134	72.3	134	250.6
4 of 4						

Marcum Creek

Unit Weight, lb/ft^3: 120 Cohesion, lbf/ft^2: 274

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	18.5	0	18.5	0	18.5			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	55	0	63	0	47	0	47	50	53
2 of 4									
3 of 4									
4 of 4									

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 51 Minimum Slope: 0.000781

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4	78.4	377.7	88.2	366.1	88.2	320.1
2 of 4						
3 of 4						
4 of 4						

Table 15
James Wolf Creek, Site 19, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 124 Cohesion, lbf/ft^2: 226

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	9	0	9	0	9			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	100	50	100	72	100	71	100	100	100
2 of 4	69	62	47	41	54	50	66	66	66
3 of 4									
4 of 4									

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 52, 79 Minimum Slope: 0.00060, 0.00070

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4	142.3	185.6	134.6	219.3	150	198
2 of 4	97.5	183	96.2	187.9	97.5	243.9
3 of 4						
4 of 4						

Table 16
Lee Creek, Site 10, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 118 Cohesion, lbf/ft^2: 356

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	17.3	0	17.3	0	17.3			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	0	0	0	0	0	0	0	0	0
2 of 4									
3 of 4									
4 of 4									

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 26 Minimum Slope: 0.001008

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4	184.6	169.2	180.8	152.5	176.9	147.8
2 of 4						
3 of 4						
4 of 4						

Table 17
Lick Creek, Site 8, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 118 Cohesion, lbf/ft^2: 356

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	17.3	0	17.3	0	17.3			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	0	0	6	0	0	0	6	12	19

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 50 Minimum Slope: 0.0011

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	86	292.7	186	214.6	188	254.6

Table 18
Long Creek, Site 20, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 130 Cohesion, lbf/ft^2: 270

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	16	0	16	0	16			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	79	0	62	0	10	0	33	33	69
2 of 4	0	0	6	0	6	0	11	17	27
3 of 4	17	0	4	0	7	4	7	14	25
4 of 4	0	0	7	0	0	0	0	0	0

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 49, 35, 54, 71 Minimum Slope: 0.0095, 0.00119, 0.00091, 0.00109

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4	100	250.9	126.5	113.8	120.4	109.6
2 of 4	162.9	135.2	217.1	108.8	197.1	141.9
3 of 4	77.8	248.6	92.6	173.8	87	286
4 of 4	105.6	80.9	67.7	289.8	85.9	246.3

Table 19
Nolehoe Creek, Site 7, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 118 Cohesion, lbf/ft^2: 356

Segment	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	17.3	0	17.3	0	17.3			
	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	5	0	6	0	12	0	19	25	31

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 31 Minimum Slope: 0.00124

Segment	1992		1993		1994	
	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	112.9	294.6	119.4	274.3	112.9	268.6

Table 20
Otoucalofa Creek, Site 14, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 122 Cohesion, lbf/ft^2: 413

Segment	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	13.3	0	13.3	0	13.3			
	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	0	0	0	0	0	0	0	0	0

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 31 Minimum Slope: 0.00124

Segment	1992		1993		1994	
	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	67.8	140.8	62.2	166.7	62.9	140.8

Table 21
Perry Creek, Site 16, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 124 Cohesion, lbf/ft^2: 177

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	22	0	22	0	22			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	NA	NA	57	0	88	0	88	88	88
2 of 4	NA	NA	32	0	62	5	69	74	74
3 of 4	NA	NA	18	5	38	0	41	41	41
4 of 4									

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 97, 64, 78 Minimum Slope: 0.00034, 0.00131, 0.00303

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4	NA	NA	71	220.5	94.8	99.7
2 of 4	NA	NA	73.4	159.8	79.7	155.7
3 of 4	NA	NA	60.3	103.1	62.8	139
4 of 4						

Table 22
Red Banks Creek, Site 9, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 124 Cohesion, lbf/ft^2: 177

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	22	0	22	0	22			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	4	0	8	0	6	0	6	13	25
2 of 4									
3 of 4									
4 of 4									

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 130 Minimum Slope: 0 .00086

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4	83.9	180	81.5	198.7	85.4	179
2 of 4						
3 of 4						
4 of 4						

Table 23
Sarter Creek, Site 15, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft³: 121 Cohesion, lbf/ft²: 331

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	14.7	0	14.7	0	14.7			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	0	0	0	0	0	0	0	0	0

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 36 Minimum Slope: 0.00093

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	113.9	160	113.9	170.9	116.7	169.9

Table 24
Sykes Creek, Site 17, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft³: 121 Cohesion, lbf/ft²: 331

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	14.7	0	14.7	0	14.7			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4 2 of 4 3 of 4 4 of 4	6	0	6	0	20	0	38	38	44

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 111 Minimum Slope: 0.00736

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4 2 of 4 3 of 4 4 of 4	67.6	211.1	73	255.4	70.3	235.1

Table 25
East Worsham Creek, Site 18a, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 125 Cohesion, lbf/ft^2: 276

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	14	0	14	0	14			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	86	24	11	0	98	20	100	100	100
2 of 4	NA	NA	NA	NA	NA	NA	NA	NA	NA
3 of 4	NA	NA	4	0	0	0	0	4	6
4 of 4									

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 74, 70 Minimum Slope: 0.000597, 0.00058

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4	67.6	319.6	67.6	273	64.9	304.9
2 of 4	NA	NA	92	184.3	72	330
3 of 4						
4 of 4						

Table 26
Middle Worsham Creek, Site 18b, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft^3: 118 Cohesion, lbf/ft^2: 233

	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	11	0	11	0	11			
Segment	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	64	6	83	0	100	0	100	100	100
2 of 4	42	0	29	0	23	0	38	38	46
3 of 4	0	0	6	0	12	0	12	12	16
4 of 4	NA	NA	0	0	3	0	8	8	8

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 63, 169, 58, 53 Minimum Slope: 0.0007, 0.0024, 0.0006, 0.0007

	1992		1993		1994	
Segment	Width	Slope	Width	Slope	Width	Slope
1 of 4	119	12.7	69.8	245.3	71.4	198.7
2 of 4	30.8	48.6	27.2	94.5	24.9	87
3 of 4	89.7	183.7	86.2	180.1	87.9	192.7
4 of 4	NA	NA	73.6	248.2	69.8	480.5

Table 27
West Worsham Creek, Site 18c, SAM and BURBANK Analysis Summary

Unit Weight, lb/ft³: 119 Cohesion, lbf/ft²: 343

Segment	1992		1993		1994		% at Risk in 1994 Friction Angle = 0 Feet of Degradation		
	Friction Angle								
	0	18	0	18	0	18			
	% at Risk		% at Risk		% at Risk		1	2	3
1 of 4	23	0	30	0	50	0	50	50	60
2 of 4	0	0	0	0	5	0	5	10	20
3 of 4	8	0	0	0	0	0	7	7	7
4 of 4	5	0	0	0	0	0	0	0	0

Percent of Width and Slope at 1994 Minimum Slope

Width at Minimum Slope, ft: 67, 63, 64, 61 Minimum Slope: 0.0008, 0.0008, 0.0008, 0.0008

Segment	1992		1993		1994	
	Width	Slope	Width	Slope	Width	Slope
1 of 4	59.7	343.5	53.7	229	50.7	338.6
2 of 4	57.1	329.9	63.5	268.8	61.9	351.3
3 of 4	48.4	556.5	70.3	299.3	79.7	218.2
4 of 4	50.3	565.7	49.2	655.3	60.7	352.3

Table 28
Subbasin Parameters, Goodwin Creek Watershed

Subbasin	Area square miles	L mles	L_{ca} mles	Lag Time hours	Average Basin Slope %
1	1.44	2.62	1.22	1.28	3.20
2	0.80	1.85	1.10	1.11	1.82
3	0.90	1.57	0 .83	0 .97	3.22
4	0 .60	1.13	0.61	0 .80	2.07
5	0 .78	1.33	0 .72	0 .89	3.70
6	1.29	1.50	0 .86	0 .97	3.23
7	0 .63	1.5	0 .74	0 .93	3.63
8	0 .46	0.76	0.48	0 .67	3.60
9	0 .86	1.39	0 .80	0 .93	3.69
10	0 .15	0 .28	0 .17	0 .36	3.73
11	0 .17	0 .31	0 .21	0 .40	2.24
12	0 .07	0 .27	0 .18	0 .36	4.71
13	0 .31	0 .68	0 .42	0.62	4.72
14	0 .14	0 .21	0 .15	0 .32	1.30

Note: L = length of the main stream channel, miles, from the outlet to the divide.
L_{ca} = length along the main channel to a point nearest the subbasin centroid.

Table 29
Channel Reach Parameters, Goodwin Creek

GISSRM Reach I.D.	Flow Routing			Sediment Routing	
	Length ft	Bed Slope ft/mile	Roughness n	Mean Size D_{50}, mm	Gradation σ, mm
1	11,486	11.82	0.037	5.714	5.988
2	6,345	39.60	0.038	(Assume Same as Reach 6)	
3	8,214	15.11	0.038	17.634	7.793
4	7,544	23.44	0. 038	(Assume Same as Reach 6)	
5	2,800	25.45	0.035	(Assume Same as Reach 6)	
6	7,150	19.94	0.038	10.510	6.292
7	5,600	25.45	0.035	(Assume Same as Reach 9)	
8	6,500	21.89	0.038	(Assume Same as Reach 9)	
9	7,600	22.58	0.040	2.053	5.537
10	5,500	41.76	0.040	(Assume Same as Reach 9)	
11	2,500	21.12	0 .040	(Assume Same as Reach 13)	
12	712	20.38	0 .040	(Assume Same as Reach 13)	
13	2,075	20.38	0 .040	4.073	7.308
14	4,150	21.65	0 .040	(Assume Same as Reach 13)	

Table 30
Sediment Yield Budget for Goodwin Creek (Based upon ARS Study for Period of Record 11/82 to 10/87)

Gages	Gaged Tons[1]	Estimated From Land Uses[1,2] tons	Gaged from Upstream Subareas[1] tons	Differences Attributed to Channel and Gully Erosion[1] tons
Peripheral				
14	7400	2200		5200
13	7400	1100		6300
12	1500	400		1100
11	1000	100		900
10	0	0		0
9	2200	200		2000
7	20500	1800		18700
6	5500	2200		3300
Nested				
8	9400	700	2500	6300
5	25200	2700	11600	10900
4	25200	2000	20500	2600
3	36100	5400	30700	-0-
2	85100	5900	76100	3100
1	140800	5700	85100	50000
Total		30400		110400

[1] Values rounded to nearest 100 tons.
[2] Excludes estimated erosion included with upstream gaged acres.

Table 31
MUSLE Coefficients for Goodwin Creek

Subbasin	Area square miles	MUSLE Coefficient C_1	MUSLE Exponent C_2
1	1.41	9.00	0.58
2	0.44	18.45	0.56
3	0.95	11.45	0.57
4	0.63	14.77	0.57
5	0.76	13.15	0.57
6	1.33	9.29	0.58
7	0.73	13.48	0.57
8	0.44	18.45	0.56
9	0.90	11.84	0.57
10	0.09	49.35	0.55
11	0.13	39.29	0.55
12	0.08	53.09	0.55
13	0.31	22.92	0.56
14	0.14	37.53	0.56

Table 32
Comparison of Observed versus Predicted Peak Flows for Gauge 1, Mouth of Goodwin Creek Watershed

Storm	Date	Duration hours	Observed Rainfall in.	Flow, cfs				
				Observed	HEC-1 (SCS)	HEC-1 (Snyder)	CASC2D	GISSRM (SCS)
1	10-17-81	3.52	2.85	1405	1532	1785	1396	1508
2	2-9-82	6.00	1.35	1000	975	1087	1046	1519
3	12-2-83	31.10	5.79	3383	1977	2139	3086	1096
4	9-30-85	22.2	2.18	158	158	185	162	21
5	12-27-88	8.6	2.34	1219	1046	1198	1218	1046

Table 33
Comparison of Water and Sediment Yield For Gauge 1, Mouth of Goodwin Creek Watershed

Storm	Date	Duration hours	Average Rainfall in.	Runoff, in.		Sediment		
				Observed	Predicted	Observed tons	Predicted tons	Difference %
1	10-17-81	3.52	2.85	0.70	0.87	1417	1470	+3.74
2	2-9-82	6.00	1.35	0.98	0.46	2335	476	-79.61
3	12-2-83	31.10	5.79	4.19	4.38	5249	4735	-9.79
4	9-30-85	22.2	2.18	0.09	0.06	77	9	-88.31
5	12-27-88	8.6	2.34	1.17	1.22	1193	1584	+32.77

Table 34
1993 Drop Structure Evaluation Summary

Label	Stream	Category[1]	Problem Type[2] a	b	c	d	e	f
HD-1	Hotophia	2			c			
LD-2	Beard's	3						
LD-1	Black	3						
LD-1	Caffe Branch	2	a			d		f
LD-1	Campbell	3						
LD-1	Caney	2	a	b		d		
LD-2	Caney	2	a	b				
LD-3	Caney	2		b				
LD-1	Crowder	2				d	e	f
LD-2	Crowder	2			c			
LD-1	Deer	1	a		c	d	e	
LD-2	Deer	2		b	c			f
LD-1	East Fork Worsham	2	a	b	c	d		f
LD-1	Eskridge	2						f
LD-2	Eskridge	2	a					
LD-1	Hickahala	2	a	b				
LD-2	Hickahala	3						
LD-3	Hickahala	2	a	b				
LD-4	Hickahala	2	a					
LD-6	Hickahala	3						
LD-7	Hickahala	3						
LD7-7	Hotophia	2	a	b		d	e	f
LD7-8	Hotophia	2	a	b		d	e	
LD-1	James Wolf	1				d	e	
LD-9B-1	Johnson Creek	2	a			d		
LD-1	Little Bogue	1	a	b	c	d	e	
LD-1	Little Mouse	3						
LD-2	Little Mouse	3						
LD-1	Long	2	a	b	c	d		
LD-2	Long	2	a					f

(Continued)

[1]Defined on page 82.
[2]Defined on page 82.

Table 34 (Concluded)								
			Problem Type					
Label	Stream	Category	a	b	c	d	e	f
LD-3	Long	2		b				f
LD-4	Long	3						
LD-5	Long	3						
LD-1	Middle Fork Worsham	2	a	b	c	d	e	f
LD-2	Middle Fork Worsham	2	a	b	c			
LD-3	Middle Fork Worsham	2			c			
LD-1	Marcum	2	a	b		d	e	
LD-1	Martin Dale	3						
LD-1	Mill	2	a					f
LD-2	Mill	3						
LD-3	Perry Creek	3						
LD-4	Perry Creek	3						
LD-1	South Fork Hickahala	2			c			
LD-2	South Fork Hickahala	3						
LD-3	South Fork Hickahala	3						
LD-1	Senatobia	3						
LD-1	Tarrey Creek	3						
LD-2	Tarrey Creek	3						
LD-1	West Fork Worsham	2	a			d		
LD-3	West Fork Worsham	2	a	b	c			
LD-4	West Fork Worsham	1			c			
LD-1	White's	2		b				
LD-1	White's	2		b				
LD-1	Worsham	2	a		c	d	e	
LD-2	Worsham	2		b				

Table 35
Assessment of Hydraulic Structures

Structure	Category	Comment
Beard's Creek low-drop structure No. 2	3	No significant problems
Black Creek low-drop structure No. 1	3	No significant problems
Caffe Branch low-drop structure No. 1	2	Has problems that should be resolved due to riprap being displaced from the face of the weir, riprap being launched at the upstream or downstream apron, and woody vegetation established in the upstream or downstream apron that is impairing the conveyance or the weir unit discharge of the structure
Campbell Creek low-drop structure No. 1	3	Has no significant problems
Caney Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap being displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, and riprap being launched at the upstream or downstream apron
Caney Creek low-drop structure No. 2	2	Has problems that should be resolved due to riprap being displaced from the face of the weir and the channel bank upstream or downstream of the structure failing
Caney Creek low-drop structure No. 3	2	Has problems that should be resolved due to the channel bank upstream or downstream of the structure failing
Crowder Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap being launched at the upstream or downstream apron, severe headcutting migrating into the basin, and woody vegetation established in the upstream or downstream apron that is impairing the conveyance or the weir unit discharge of the structure
Crowder Creek low-drop structure No. 2	2	Has problems that should be resolved due to bank erosion or piping beneath the riprap caused by overbank drainage
Deer Creek low-drop structure No. 1	1	Under an imminent threat of loss of function due to riprap displaced from the face of the weir, bank erosion or piping beneath the riprap caused by overbank drainage, riprap being launched at the upstream or downstream apron, and severe headcutting migrating into the basin

(Sheet 1 of 5)

Table 35 (Continued)

Structure	Category	Comment
Deer Creek low-drop structure No. 2	2	Has problems that should be resolved due to the channel bank upstream or downstream of the structure failing and bank erosion or piping beneath the riprap caused by overbank drainage
East Fork Worsham Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, bank erosion or piping beneath the riprap caused by overbank drainage, riprap launched at the upstream or downstream apron, and woody vegetation established in the upstream or downstream apron that is impairing the conveyance or the weir unit discharge of the structure
Eskridge Creek low-drop structure No. 1	2	Has problems that should be resolved due to woody vegetation established in the upstream or downstream apron that is impairing the conveyance or the weir unit discharge of the structure
Eskridge Creek low-drop structure No. 2	2	Has problems that should be resolved due to riprap displaced from the face of the weir
Hickahala Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap being displaced from the face of the weir and the channel bank upstream or downstream of the structure failing
Hickahala Creek low-drop structure No. 2	3	Has no significant problems
Hickahala Creek low-drop structure No. 3	2	Has problems that should be resolved due to riprap being displaced from the face of the weir and the channel bank upstream or downstream of the structure failing
Hickahala Creek low-drop structure No. 4	2	Has problems that should be resolved due to riprap being displaced from the face of the weir
Hickahala Creek low-drop structure No. 6	3	Has no significant problems
Hickahala Creek low-drop structure No. 7	3	Has no significant problems
Hotophia Creek high-drop structure No. 2	2	Has problems that should be resolved due to bank erosion or piping beneath the riprap caused by overbank drainage
Hotophia Creek low-drop structure No. 7-7	2	Has problems that should be resolved due to riprap displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, riprap launched at the upstream or downstream apron, severe headcutting migrating into the basin, and woody vegetation established in the upstream or downstream apron that is impairing the conveyance or the weir unit discharge of the structure

(Sheet 2 of 5)

Table 35 (Continued)		
Structure	**Category**	**Comment**
Hotophia Creek low-drop structure No. 7-8	2	Has problems that should be resolved due to riprap displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, riprap launched at the upstream or downstream apron, and severe headcutting migrating into the basin
James Wolf Creek low-drop structure No. 1	1	Under an imminent threat of loss of function due to riprap launched at the upstream or downstream apron and severe headcutting migrating into the basin
Johnson Creek low-drop structure No. 9B-1	2	Has problems that should be resolved due to riprap being displaced from the face of the weir and riprap being launched at the upstream or downstream apron.
Little Bogue Creek low-drop structure No. 1	1	Under an imminent threat of loss of function due to riprap displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, bank erosion or piping beneath the riprap caused by overbank drainage, riprap launched at the upstream or downstream apron, and severe headcutting migrating into the basin
Little Mouse Creek low-drop structure No. 1	3	Has no significant problems
Little Mouse Creek low-drop structure No. 2	3	Has no significant problems
Long Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, bank erosion or piping beneath the riprap caused by overbank drainage, and riprap launched at the upstream or downstream apron
Long Creek low-drop structure No. 2	2	Has problems that should be resolved due to riprap being displaced from the face of the weir and woody vegetation established in the upstream or downstream apron that is impairing the conveyance or the weir unit discharge of the structure
Long Creek low-drop structure No. 3	2	Has problems that should be resolved due to the channel bank upstream or downstream of the structure failing and woody vegetation established in the upstream or downstream apron that is impairing the conveyance or the weir unit discharge of the structure
Long Creek low-drop structure No. 3	3	Has no significant problems
Long Creek low-drop structure No. 4	3	Has no significant problems

(Sheet 3 of 5)

Table 35 (Continued)

Structure	Category	Comment
Middle Fork Worsham Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, bank erosion or piping beneath the riprap caused by overbank drainage, riprap launched at the upstream or downstream apron, severe headcutting migrating into the basin, and woody vegetation established in the upstream or downstream apron that is impairing the conveyance or the weir unit discharge of the structure
Middle Fork Worsham Creek low-drop structure No. 2	2	Has problems that should be resolved due to riprap displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, and bank erosion or piping beneath the riprap caused by overbank drainage
Middle Fork Worsham Creek low-drop structure No. 3	2	Has problems that should be resolved due to bank erosion or piping beneath the riprap caused by overbank drainage
Marcum Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, riprap launched at the upstream or downstream apron, and severe headcutting migrating into the basin
Martin Dale Creek low-drop structure No. 1	3	Has no significant problems
Mill Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap displaced from the face of the weir and woody vegetation established in the upstream or downstream apron that is impairing the conveyance or the weir unit discharge of the structure
Mill Creek low-drop structure No. 2	3	Has no significant problems
Perry Creek low-drop structure No. 3	3	Has no significant problems
Perry Creek low-drop structure No. 4	3	Has no significant problems
South Fork Hickahala Creek low-drop structure No. 1	2	Has problems that should be resolved due to bank erosion or piping beneath the riprap caused by overbank drainage
South Fork Hickahala Creek low-drop structure No. 2	3	Has no significant problems
South Fork Hickahala Creek low-drop structure No. 3	3	Has no significant problems
Senatobia Creek low-drop structure No. 1	3	Has no significant problems

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Table 35 (Concluded)

Structure	Category	Comment
Tarrey Creek low-drop structure No. 1	3	Has no significant problems
Tarrey Creek low-drop structure No. 2	3	Has no significant problems
West Fork Worsham Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap displaced from the face of the weir and riprap launched at the upstream or downstream apron
West Fork Worsham Creek low-drop structure No. 3	2	Has problems that should be resolved due to riprap displaced from the face of the weir, the channel bank upstream or downstream of the structure failing, and bank erosion or piping beneath the riprap caused by overbank drainage
West Fork Worsham Creek low-drop structure No. 4	1	Under an imminent threat of loss of function due to bank erosion or piping beneath the riprap caused by overbank drainage
White's Creek low-drop structure No. 1	2	Has problems that should be resolved due to the channel bank upstream or downstream of the structure failing
Worsham Creek low-drop structure No. 1	2	Has problems that should be resolved due to riprap displaced from the face of the weir, bank erosion or piping beneath the riprap caused by overbank drainage, riprap being launched at the upstream or downstream apron, and severe headcutting migrating into the basin.
Worsham Creek low-drop structure No. 2	2	Has problems that should be resolved due to the channel bank upstream or downstream of the structure failing

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Thomas J. Pokrefke, Nolan K. Raphelt, David L. Derrick, Billy E. Johnson, Michael J. Trawle, Chester C. Watson

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180-6199
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13. ABSTRACT *(Maximum 200 words)*
The purpose of monitoring the Demonstration Erosion Control (DEC) Project is to evaluate and document watershed response to the implemented DEC Project. Documentation of watershed responses to DEC Project features will allow the participating agencies a unique opportunity to determine the effectiveness of existing design guidance for erosion and flood control in small watersheds. The monitoring program includes 11 technical areas: stream gauging, data collection and data management, hydraulic performance of structures, channel response, hydrology, upland watersheds, reservoir sedimentation, environmental aspects, bank stability, design tools, and technology transfer.

This report includes detailed discussion of the eight technical areas that were investigated by the U.S. Army Engineer Waterways Experiment Station during Fiscal Year 1994, i.e., all of these areas except upland watersheds, reservoir sedimentation, and environmental aspects.

In the area of data collection and data management, installation of continuous stage gauge instrumentation at 33 sites and crest gauges at an additional 42 sites was completed and data collection initiated. The initial development of the engineering database on Intergraph workstations was completed and made available to the U.S. Army Engineer District, Vicksburg, for testing.

(Continued)

14. SUBJECT TERMS

Channel degradation	Hydraulic data collection
Engineering database	Hydrologic modeling
Erosion control	Sedimentation

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In the area of hydraulic performance of structures, a model study to determine the feasibility of a low-drop structure using a 10-ft drop was conducted. Selected high- and low-drop structures were instrumented with stage gauges. The stage data will be used in calculating discharge coefficients for rating curves.

In the area of channel response, the first detailed topographic survey of the 20 long-term sites was completed. The initial broad-based geomorphic studies of 1 watersheds and detailed geomorphic studies of 3 watersheds were completed.

In the area of hydrology, development of HEC-1 hydrology models for 10 watersheds was initiated. The evaluation of the CASC2D hydrology model using the Goodwin Creek watershed was initiated.

In the area of bank stability, a model study to determine the applicability of the bendway weir concept for bank stabilization was conducted.

In the area of design tools, a riser pipe design system housed on the engineering database (Intergraph) was developed, tested, and made available for District use on the Coldwater River watershed.

In the area of technology transfer, a video report on the DEC Project was completed, and a second video report on channel degradation processes was initiated.